ARCHITECTURAL CONSTRUCTION

ARCHITECTURAL CONSTRUCTION

VOLUME TWO

AN ANALYSIS OF THE STRUCTURAL DESIGN OF AMERICAN BUILDINGS

BY

WALTER C. VOSS, S. B.

CONSULTING ENGINEER, BOSTON, MASSACHUSETTS
FORMERLY HEAD OF DEPARTMENT OF ARCHITECTURAL CONSTRUCTION
WENTWORTH INSTITUTE, BOSTON, MASSACHUSETTS

AND

EDWARD A. VARNEY, S. B.

STRUCTURAL ENGINEER, CAMBRIDGE, MASSACHUSETTS
FORMERLY INSTRUCTOR OF MECHANICS AND STRUCTURAL DESIGN
WENTWORTH INSTITUTE, BOSTON, MASSACHUSETTS

BOOK TWO

STEEL CONSTRUCTION

NEW YORK
JOHN WILEY AND SONS, INC.
LONDON, CHAPMAN AND HALL, LIMITED
1922

WITH GRATITUDE AND AFFECTION
TO OUR MOTHERS
T. V., AND G. V.,
WHO AFFORDED US OUR OPPORTUNITIES
AND
OUR WIVES
L. M. V., AND M. L. V.,
WHO SO CONTINUOUSLY ENCOURAGED OUR EFFORTS

PREFACE TO VOLUME TWO

In the construction of all our modern buildings of any prominence, and indeed even in some of our simpler structures, the mind trained in engineering principles works in close harmony with the mind evolving the architecturally serviceable and beautiful. In our more pretentious buildings, involving the use of complicated designs in wood, steel or concrete, the architect relies almost entirely upon the engineer for all structural advice. A sympathetic knowledge of the principles underlying the other's field by each party to this partnership is indispensable, not only from the viewpoint of economy and serviceability, but also because of the broadening influence which such a knowledge lends each in appreciating both the architectural and structural limitations in juxtaposition.

The architect who aims to become reasonably well-versed in the fundamental principles of structural design and detail will not so arrange a plan as to penalize the construction, but will be able to sense the usual economic principles and govern his layout accordingly. Likewise, the engineer should become as well versed in the general practice of the architect as possible, in order that he may wisely guide those features of the design which seriously affect his efforts toward economy, without any sacrifice in the beauty or serviceability of the building.

It is with the object of bringing about this closer coöperation that this volume is written. An attempt has been made to outline in a simple fashion those features of engineering which an architect might well command as a part of his general knowledge. In presenting these, they have been closely tied into correct architectural and engineering practice, so that not only the architect but also the engineer may profit thereby.

The volume has been subdivided into five books, as follows:

Book One — Wood Construction
Book Two — Steel Construction
Book Three — Concrete Construction
Book Four — Walls and Foundations
Book Five — The Mechanics of Structural Design.

The reasons for so dividing the volume are undoubtedly obvious. Each of these phases of engineering practice are often specialties and with this in mind the authors have made each a separate entity. Combined, it is hoped they will reflect a composite of the modern practice of structural engineering as applied to Architectural Construction.

The authors have striven diligently to emulate the latest and best practice in actual construction and have included as many recent examples of such practice as possible for the purpose of illustration. In all cases each principle of structural design has been analyzed so that its practical application, and therefore its incident interest to the architect and engineer would be self-evident. The relation of this principle to the entire building and to those architectural and structural details which control it has been studiously considered. This principle has been brought to the mind of the reader by practical illustrations so that he might analyze it and so that its fundamental, theoretical conditions might be clear.

Each step has been covered by illustrative problems and followed by correlative problems for solution. The principle has then been tied into the major body of structural design and detail in its proper chronological order, so that it then might serve as a basis for the study of the next step.

In attempting a subject so laden with mathematical data and symbols, it would be fortunate indeed if no errors were made. The present status of engineering opinion shows many different points of view. The authors have humbly expressed their opinion, based upon their best knowledge and judgment. Again, they lay no claim to have exhausted so comprehensive and so ever-changing a field. They will be glad to receive criticisms from any source, of possible errors, which may tend to make future editions of the volume more useful to the profession.

The authors wish to take this opportunity to express their gratitude, and sincere appreciation to Messrs. Leonard E. Goguen and Carl M. Stiles for their faithfulness and skill in aiding in the preparation of illustrative matter. Our gratitude is due also to several architects, engineers, and products companies, who have loaned photographs and drawings for purposes of illustration. These are acknowledged at various points throughout the volume.

PREFACE TO BOOK TWO

This subdivision of Volume Two, called Book Two, deals with the principles and practice of design and detail in structural steel. It is assumed that the reader has a working knowledge of structural mechanics. Many other materials, necessary in the steel frame, are of necessity included and are discussed from the point of view of load delivery in as practical a way as possible.

This book, then, deals primarily with the problems which confront the designer of buildings which are framed essentially with structural steel

CONTENTS

PART I

DESIGN OF BEAMS

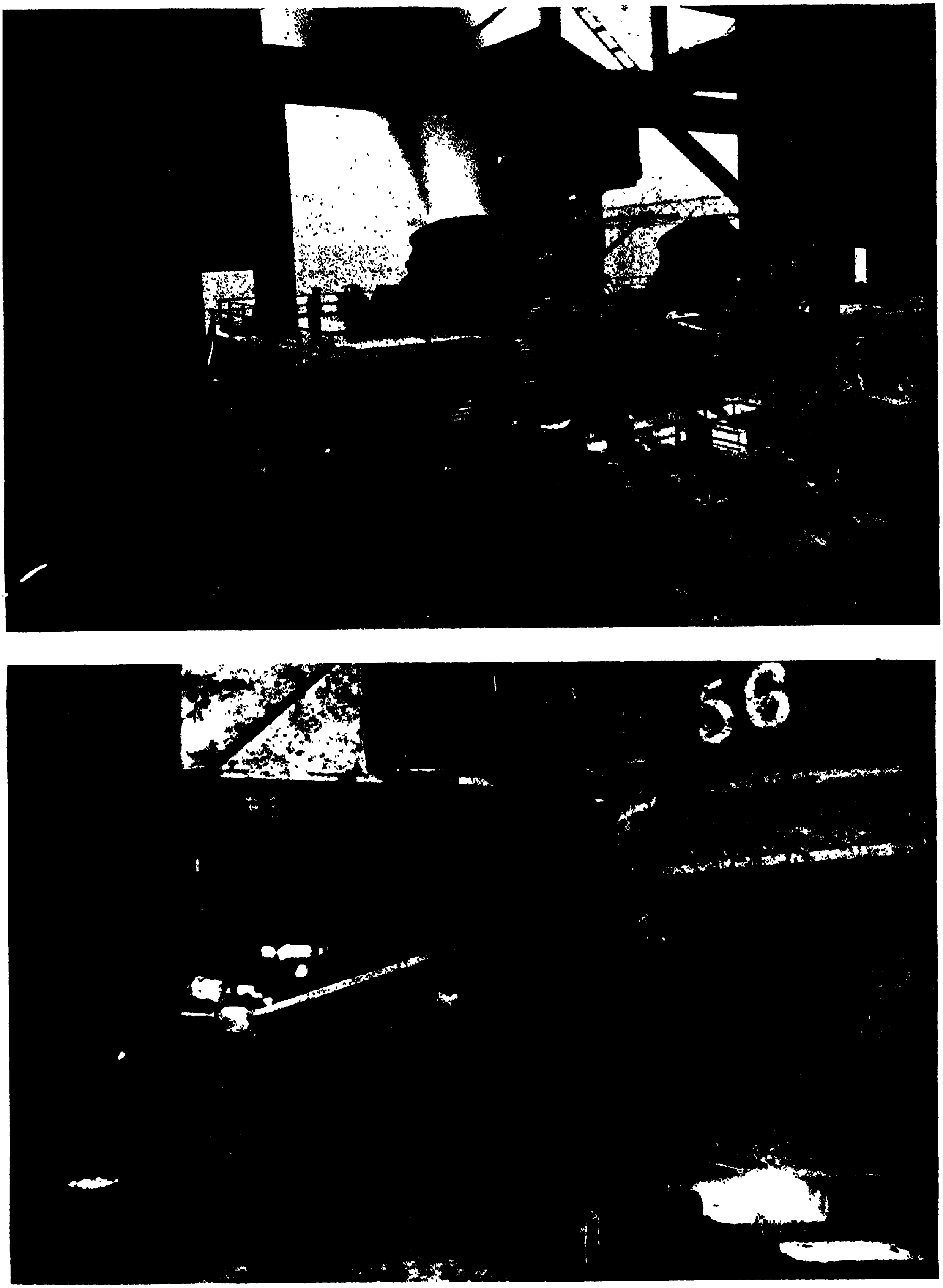
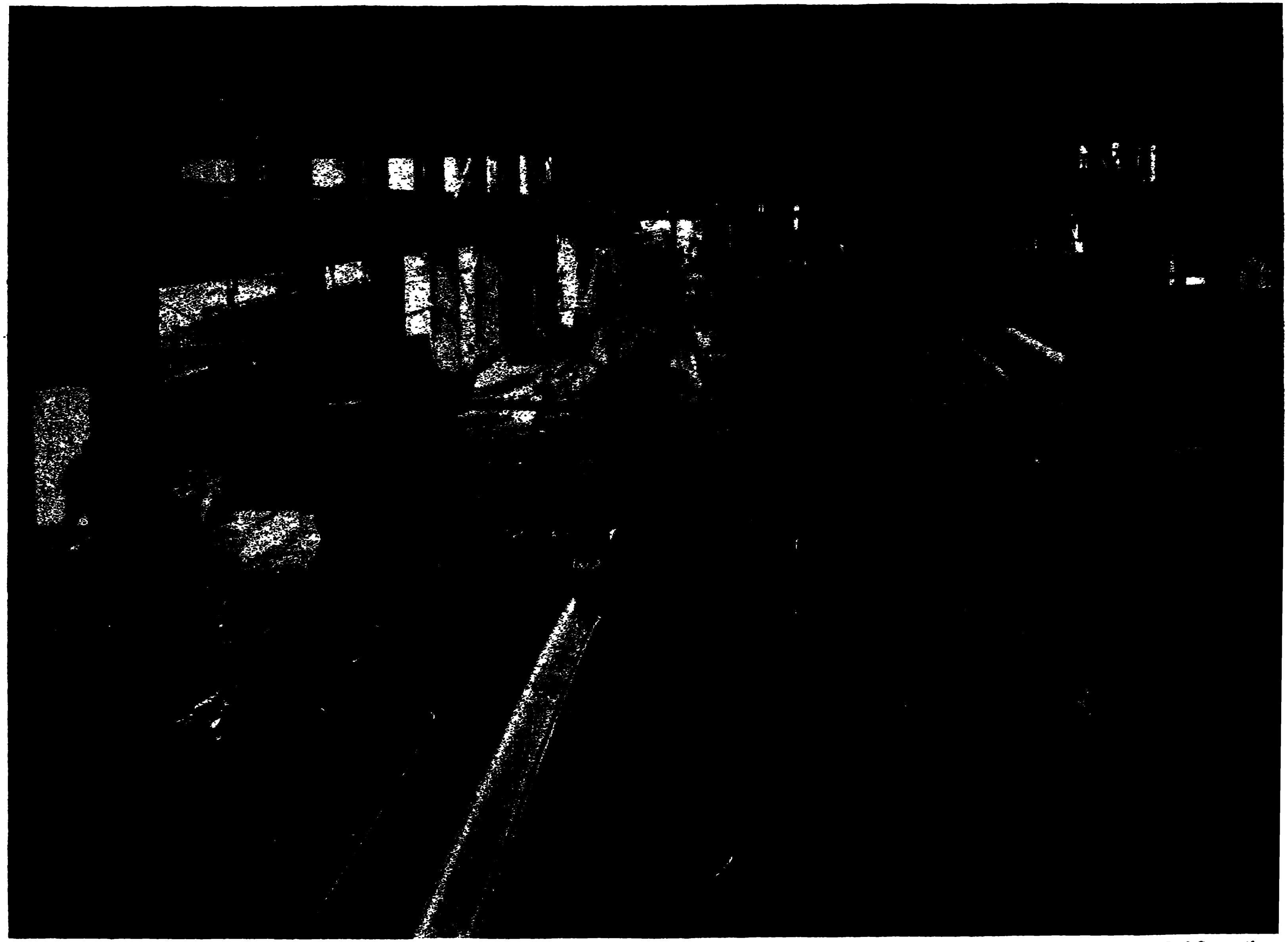


PLATE 1 TOP — A BESSEMER CONVERTOR IN ACTION
BOTTOM — POURING A HEAT

Courtesy of the United States Steel Corporation



Courtesy of the United States Steel Corporation

PLATE 2 A TYPICAL STRUCTURAL MILL
TABLES AND ROLLS WITH BEAM IN PASS

PLATE 3 A TYPICAL STOCK YARD

Courtesy of the New England Structural Company

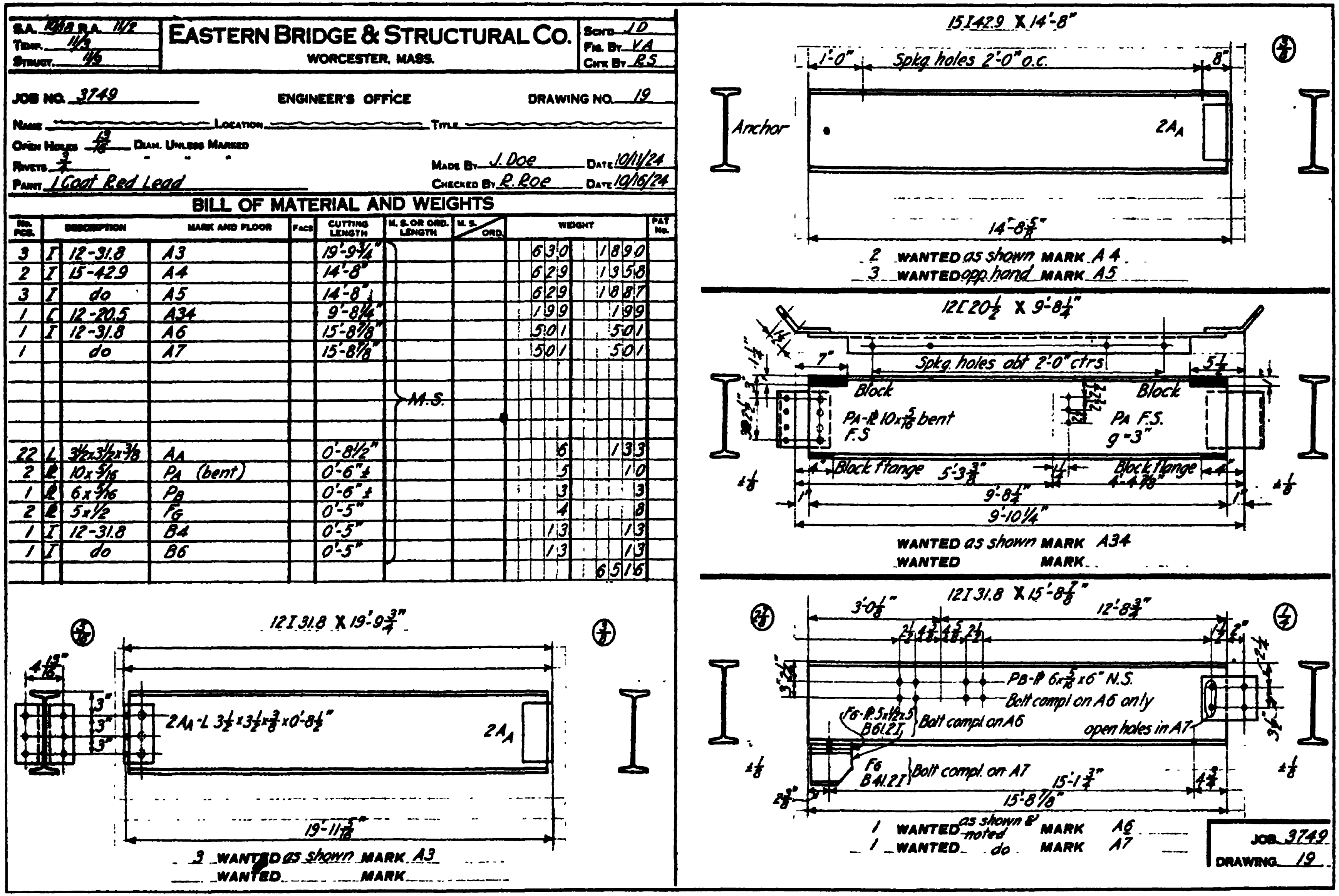


PLATE 4 A TYPICAL SHEET OF BEAM DETAILS

Courtesy of the Eastern Bridge and Structural Company

PLATE 5 TYPICAL TEMPLATE SHOP

Courtesy of the New England Structural Company

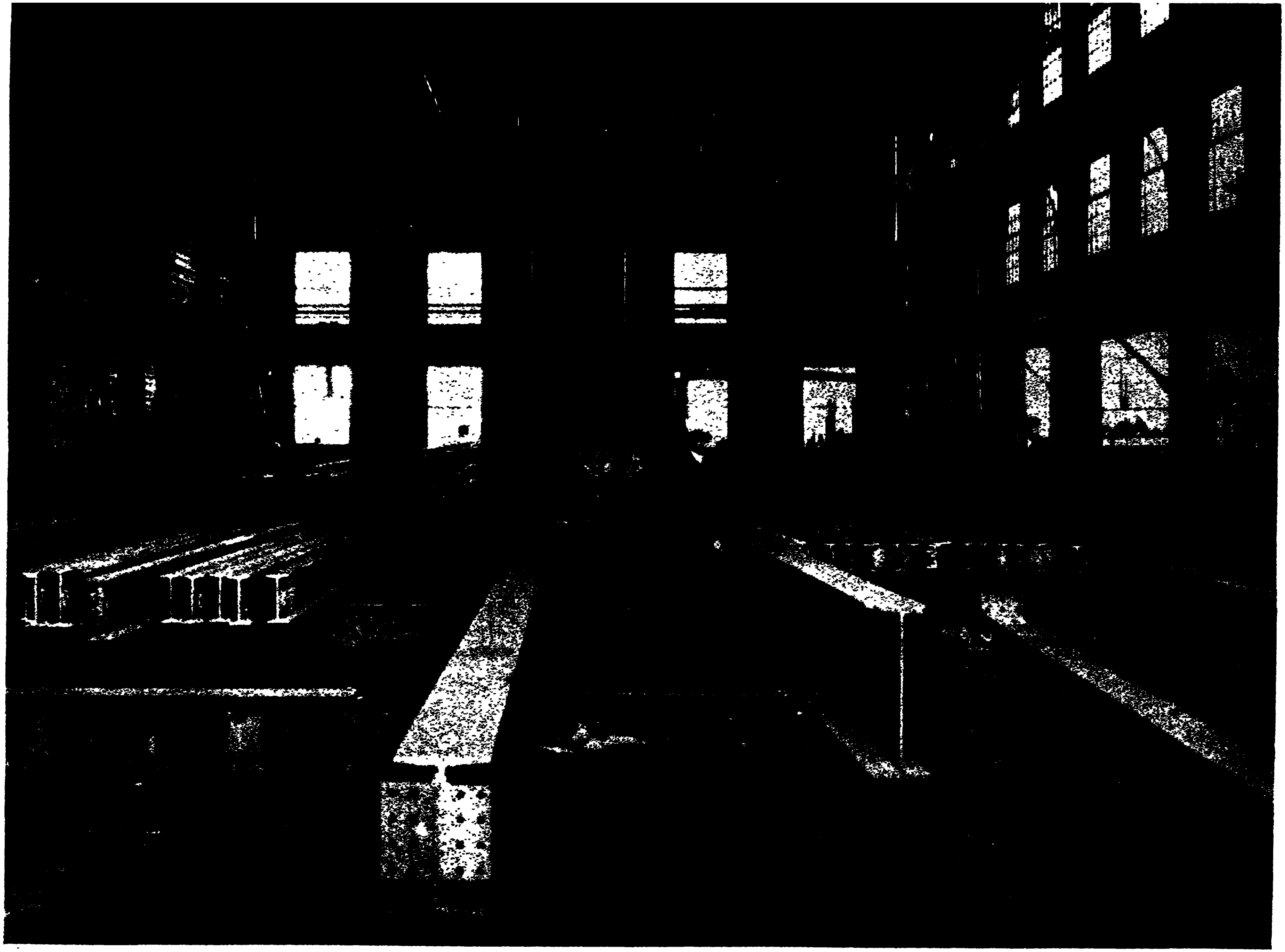


PLATE 6 TYPICAL BEAM FABRICATING SHOP

Courtesy of the New England Structural Company

PLATE 7 A TEST OF A STEEL I BEAM
CHARACTERISTIC FAILURES

CHAPTER 1

STRUCTURAL STEEL SHAPES

1. Manufacture of Iron and Steel.*

The manufacture of iron and steel is accomplished in two definite steps, the first of which is that of extracting the metal from its ore, and the second is a treatment of the metal by adding varying proportions of different elements to give it the desired qualities. The origin of cast iron, wrought iron and steel, in their various grades, is **iron ore** (hematite principally), which is essentially a combination of about one-half iron and one-half "gangue" material (usually silica, clay, etc.). About four-fifths of the annual tonnage of this ore is mined from open pits in the Lake Superior region. It is shipped to the centers where fuel is cheap, principally in the Pittsburgh district, which is also the important center of the steel industry. Usually no preliminary treatment of the ore is made, although in special cases it may be washed or roasted to remove certain ingredients.

The first step in the manufacture is that of **smelting** the ore in a blast furnace. This is a process of reducing the metal from its chemical combination with the mineral, by fusing the iron with its gangue material to a slag, so that the metal may be recovered. Coke is the common fuel used, and by its carbon content it serves as a reducing agent for the oxides in the iron ore. A flux, usually limestone, is added to the charge already containing the fuel and the ore, to insure fluidity. This also aids in drawing off the extraneous materials, forming the slag. The furnace is charged at the top and is run as a continuous operation. The molten metal flows by the action of gravity down to the bottom of the furnace, where it is tapped off every six hours, and it is cast into moulds which form bars weighing about 100 pounds each. The metal at this stage is called **pig iron.** It may be of different grades, depending upon the use to which it is to be put. Pig iron contains 3 to 4% carbon, 1 to 3% silicon, 0.03 to 0.12% sulphur, 0.05 to 1.0% phosphorus, and 0.5 to 1.0% manganese, the amounts depending upon the grade of the metal. For economic reasons, practically all iron ore is first reduced to pig iron, but the requirements of refined iron and steel call for reducing the elements mentioned above, particularly the carbon, silicon and phosphorus, to lower limits.

Cast iron is made by remelting pig iron, scrap iron, and a small amount of flux in a cupola furnace. Little change is made in the proportion of the elements, and the molten metal is poured into moulds of varying sizes and shape. Cast iron is of two kinds, namely **gray** and **white.** The kind is determined by the rate of cooling and by the varying amounts of the elements contained. Gray cast iron has the larger contents of carbon, silicon and sulphur. It forms the bulk of the cast iron made, while white cast iron is used for special instances and also as a base for **malleable** cast iron. The use of cast iron in the building industry is now much less than that of former years, because of the reliability of steel, and its use is limited to some types of column bases and caps, pipe, and occasional lintels and columns.

Wrought iron is made by heating pig iron with an oxide of iron and a flux in a reverberatory furnace until it solidifies into a pasty mass which is called a puddle ball. The latter is heated to a welding temperature and the fluid slag is removed by a squeezer, after which the remainder is welded into a bloom. This is then sent through a train of roughing rolls and converted into flat muck bars about 2″ to 4″ thick. A large proportion of the wrought iron made in this country is made, not by refining pig iron, but by heating and rolling scrap iron. This is called (1) busheled iron, from which sheets are made, (2) fagoted iron, and (3) charcoal iron, which is the base for crucible steel, depending upon the amount of scrap added. Muck bars are reheated, welded under a hammer, and then rolled into sheets and merchant bars. The result is wrought iron. Structural shapes were once rolled from this material but this practice has been discontinued. In present practice, welded pipe, sheet metal, blacksmith stock, bolts and fastenings, and so on, are often made of wrought iron, and this material is also a base for high-grade tool steels. (See also Art. 850, Vol. I.)

Steel is made by two common processes. These are identified as:

Bessemer (Pneumatic) $\left\{\begin{array}{l}\text{Acid}\\ \text{Basic (Thomas-}\\ \quad\text{Gilchrist)}\end{array}\right\}$ and

Open-hearth (Siemens-Martin) $\left\{\begin{array}{l}\text{Acid}\\ \text{Basic}\end{array}\right\}$

The processes differ principally in the type of apparatus used to make the conversion. The acid and basic metals, which may be made by either process, differ principally in the chemical reactions set up by the lining used in the furnace. The

* For a more detailed study, see Johnson's "Materials of Construction," and Mills' "Materials of Construction" — John Wiley & Sons, Inc.

acid lining does not remove the phosphorus from the metal, while the basic lining does. Since an excess amount of phosphorus is undesirable in the usual kinds of steel, the basic steel is the most common. It is now generally conceded that bessemer steel is inferior in quality and less reliable than open-hearth steel, so that a **large proportion** of the steel employed in present practice **is** made by the **basic open-hearth** method.

SPECIFICATION CLAUSES*

Process (*a*) Structural steel, except as noted in par. (*b*), may be made by the bessemer or the open-hearth process.

(*b*) Rivet steel, and steel for plates or angles over $\frac{3}{4}$ inch in thickness which are to be punched, shall be made by the open-hearth process.

Describing the open-hearth process in general, the pig iron is charged into a furnace with a shallow hearth, along with iron ore, steel scrap and limestone, and a hot air blast is passed over it to form a gas. The purpose of the limestone as a flux is to form a blanket to prevent the oxygen in the air from uniting except as desired. The impurities in the iron become oxidized by the iron ore which in turn receives new oxygen from the air. In this manner the percentages of carbon and phosphorus are reduced so that the metal becomes more workable. The amounts of carbon, phosphorus and sulphur are very important in the manufacture of steel, and it is by scientific control that this material is made so dependable. Thus, steel is classified into three general grades by the amounts of the carbon content — namely, soft or mild, intermediate or medium, and high carbon or hard. Two per cent of carbon marks the theoretical dividing line between steel and cast iron, but all steel generally contains less than 0.9% of carbon. Any increase in carbon content increases the strength of the steel, but it decreases the ductility, softness and resistance to shocks. The percentages of carbon which are established for the various kinds of steel are the results of experience, and they are controlled according to the commercial use of the metal. Thus in structural steel, an increase of strength is desirable but with not too great a loss in ductility (for rolling) and softness (for fabrication). Rivet steel has less carbon content so that the rivets may be soft and thus easily driven, while rail steel has a higher carbon content than structural steel, as little fabrication is required and considerable resistance to shocks and wear is desirable. Table 1 gives the approximate carbon content for the more important steels used in construction. Excess amounts of phosphorus and sulphur are injurious to the steel:

Cold Short "Phosphorus tends to cause the formation of coarse, crystalline structure during cooling of steel. Because of this, or for other reasons, the cooled steel, although its static strength may be somewhat increased, yields more easily to shocks and is unsafe as structural material, that is, it is 'cold short.' "†

Red Short "Sulphur makes steel 'red short'; that is, it causes it to crack when rolled hot. It also makes welding difficult. It certainly would be desirable to specify sulphur as low as possible without hardship to the steel manufacturer."†

TABLE 1‡
PROPERTIES OF VARIOUS STEELS

Classification Based on			Per Cent Carbon	Tensile Strength (lb/in.²)	Per Cent Elongation in	
Usage	Hardness	Manufacture			8 in.	2 in
Rivet	Extra soft	O. H.	0 08–0 15	45– 55,000	30	
Tubing and pressed metal .	Extra soft	O. H.	0 08–0 15	45– 55,000	30	
Screw stock .		O. H / Bess.	0 10–0 20	55– 65,000	25	
Boiler plate	Mild or Soft	O. H.	0 10–0 20	55 · 65,000	25	
Structural ...		O. H. / Bess.	0 10–0 20	55– 65,000		
Structural.....	Medium	O. H.	0 20–0 35	65– 75,000	22	
Machine	Medium	O. H.	0 20–0 30	60 70,000	23	
Rail..........	Hard	O. H. / Bess.	0 40–0 75	85–125,000	..	10–15
Tire..........	Hard	O. H.	0 50–0 80	110–130,000	..	10
Tool..........	Hard	O. H.	0 85–1 25	110 130,000	..	8

For the above reasons, the amounts of phosphorus and sulphur are limited. Thus for structural steel the following specification is used:

SPECIFICATION CLAUSE*

Chemical Composition The steel shall conform to the following requirements as to chemical composition:

		Structural Steel	Rivet Steel
Phosphorus	Bessemer	not over 0.10%	....
	Open-hearth	not over 0.06%	not over 0.06%
Sulphur			not over 0.045%

The molten steel is run off from the furnace and cast into **ingots** which are usually from 12″ to 20″ square and about 6′-0″ long. The steel in this form is mild steel, which is the grade of the majority of structural steel. It may be reheated and recarbonized by adding the proper percentages of carbon to make higher grades of steel. By introducing other elements such as chromium, nickel,

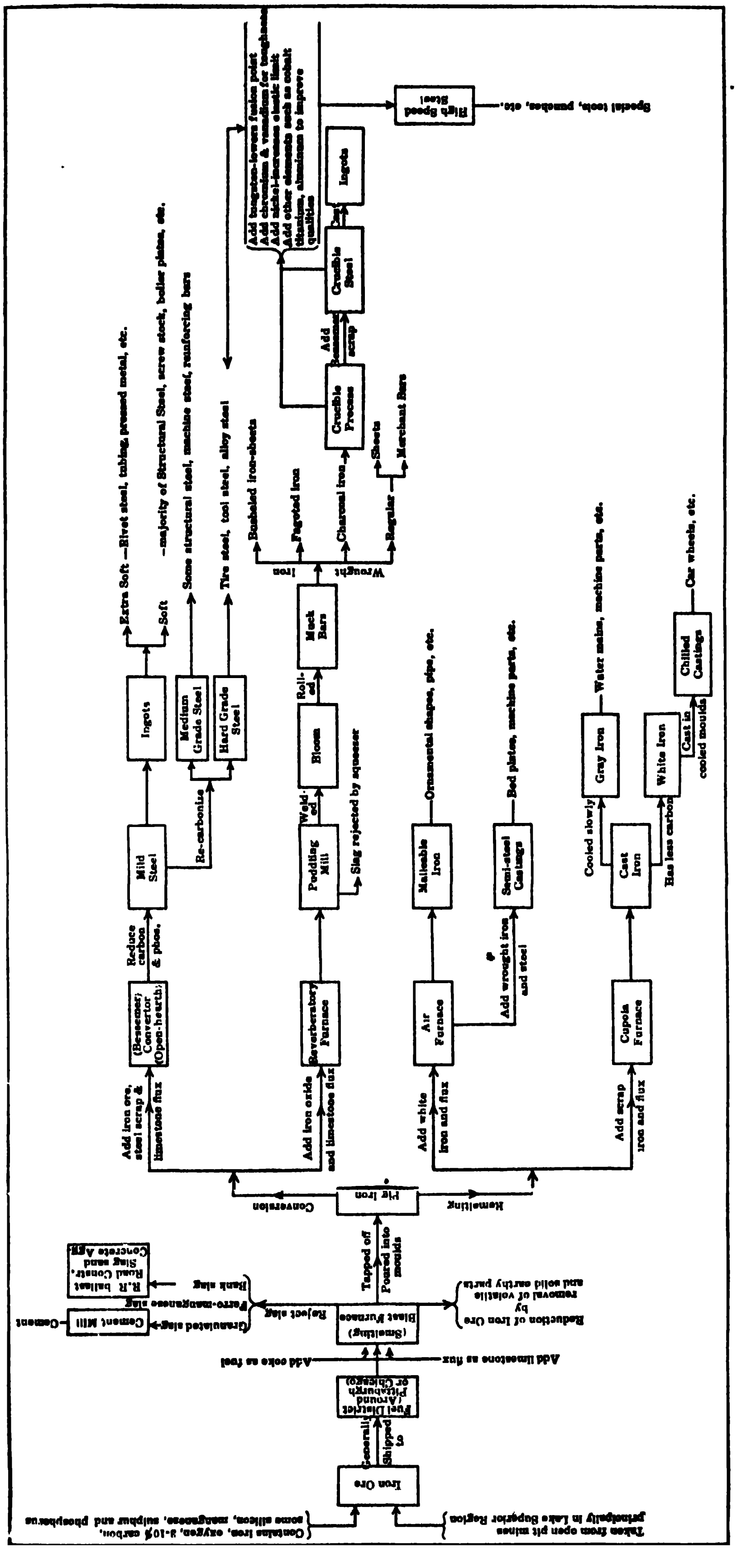
* American Society for Testing Materials, Standard Specifications for Structural Steel for Buildings, Serial Designation, A9-14.

† From "Materials of Machines" by A. W. Smith — John Wiley & Sons, Inc.

‡ From p. 621, Johnson's "Materials of Construction," John Wiley & Sons, Inc.

TABLE 2

CHART OF IRON AND STEEL MANUFACTURE



vanadium, and so on, alloy steels are made. Variations in the manufacture are also made with duplex, crucible, or electric furnaces. Table 2 is given to make a brief summary of the industry.

2. Rolling the Shapes.

After the molten steel, as it comes from the convertor, is cast into ingots (Art. 1), it may be rolled into definite shapes which are used in the construction industry. When the cast-iron moulds are stripped, the ingots cannot be rolled immediately because the temperatures of the outside and inside portions are not the same. If they were rolled directly after the stripping, the fluid metal inside would be forced out of place, while ingots which were cooled so that the inside had solidified, would have their exteriors too hard for rolling. Consequently the ingots are first put into soaking pits, which are a form of gas-fired furnace, and reheated to a temperature which will facilitate easy rolling. They are then passed through a series of blooming (or cogging) rolls and the cross-section is reduced to about one-half the size of the original ingot. Figure 1 shows Three-High I Beam Rolls. Those above are called roughing rolls; and those below, finishing rolls. The beam passes through a set of rolls between these which is called an intermediate roll stand. The general method of rolling is that of drawing the ingot through two revolving rolls in the direction of their rotation. An upward pressure, a downward pressure, and a pull are thus exerted on the ingot, and the section is reduced to a thickness equal to the distance between the rolls, is increased in width slightly due to bulging, and is greatly increased in length. This is then rolled on edge, and a square section results, the sides of which are equal to the distance between the rolls. These operations are repeated with decreasing distances between the rolls until the desired section is obtained.

The number of steps, called passes, are scientifically determined so that the reduction is made with the maximum speed and efficiency, but also to shape the section uniformly on all sides, so that the advantages resulting from the mechanical treatment of the molecules will be obtained. The resulting shapes are called blooms or **billets.**

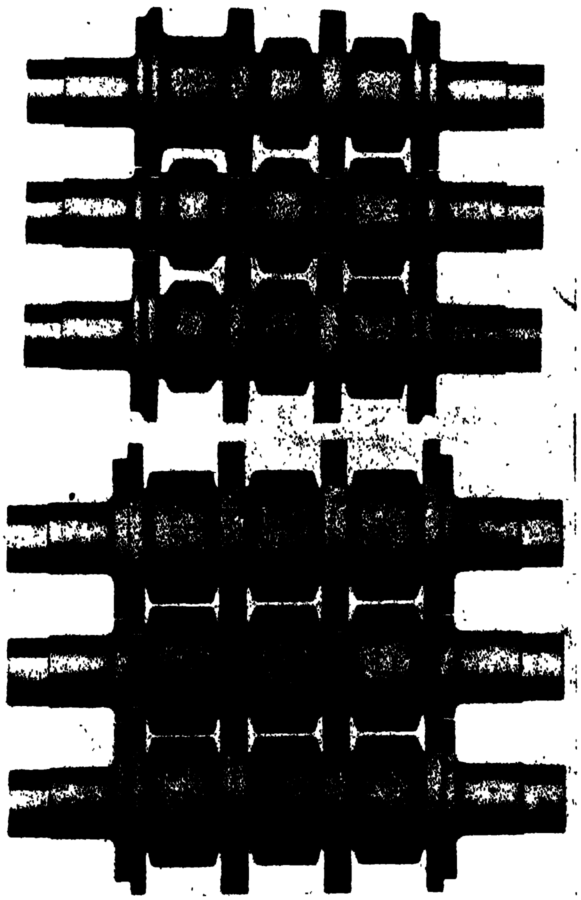


Fig. 1

Sections of the desired dimensions are made from the billets by a continuation of the rolling. For example, billets are rolled into thick rectangular sections which are called **slabs.** These may be reheated and rolled by successive passes into **plates.** After cooling, the latter may be straightened and sheared to size. Certain sizes of plates are made in **universal mills** which have, in addition to the usual horizontal rolls, a set of small rolls with their axes vertical. Such plates are rolled to their exact widths. Billets are also reduced by a series of rollings to flat bars (**flats**), **square bars,** and **round rods,** and by special equipment, **deformed bars** are made.* Round rods may be pickled in a solution of sulphuric acid to remove the scale, washed, and then cold drawn through a series of dies and annealed, to produce **wire.** **Sheets** are produced in a manner similar to plates in that the billets are first rolled into sheet bars about $\frac{1}{2}'' \times 8''$ and then cut to a length equal to the width of sheet desired. These trimmed sheet bars are then heated and rolled separately in pairs on a two-high mill until quite a reduction has been effected. The two pieces are then rolled together until the loss of heat makes it impossible to further reduce them. Four sheets are then doubled in a pack and stamped flat. After reheating, the eight leaves are rolled to gauge and later trimmed and annealed. They are then ready for the commercial surface coating desired. Billets may also be rolled to a round shape, pierced, and then drawn over a mandrel. The resulting material is made into **pipe** and **tubing.**

The so-called **structural shapes** are made from the billets by a series of rolls that have special grooves in them which vary according to the type of the product (Fig. 1). They are first sent through **roughing rolls** to approximate the shape, then through **intermediate roll stands,** and finally through the **finishing rolls** to obtain the exact dimensions. Figure 2 shows the changes in shape during the rolling of (*a*), a 100# rail section, and (*b*), a $1\frac{1}{4}'' \times 1\frac{1}{4}''$ angle. Figure 3 shows some of the structural shapes rolled, together with the usual trade names of the parts. Other shapes that are rolled include ship building channels, bulb angles and bulb tees; car building bulb angles, railroad construction ties, rails, splices and so on; sheet piling, trough, cor-

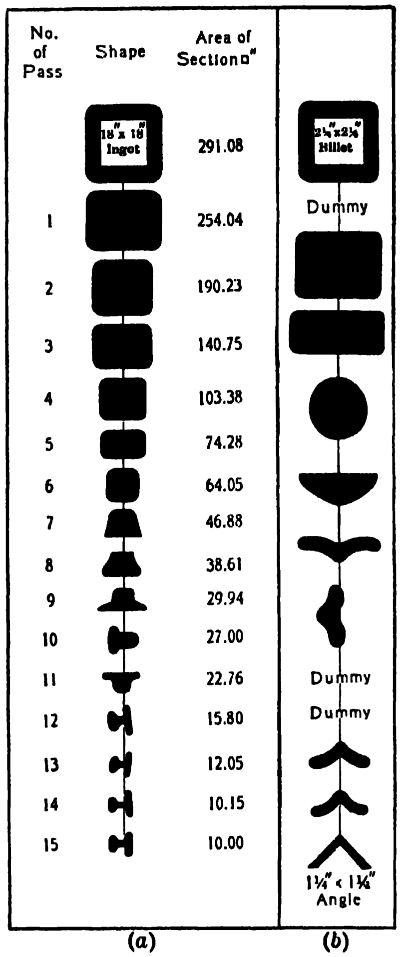


Fig. 2†

* Many special sections are included in these, such as square edge, nut steel, and round edge flats, skelp, and round cornered squares, half rounds, hexagons, and octagons. See Carnegie Pocket Companion.

† Adapted from the Iron Age, Vol. 92, pp. 968 and 1037.

rugated and checkered plates.* The **common shapes** which are used in building construction are **I beams, channels, plates** and **angles.** Tees are also used to some extent in stair and roof framing

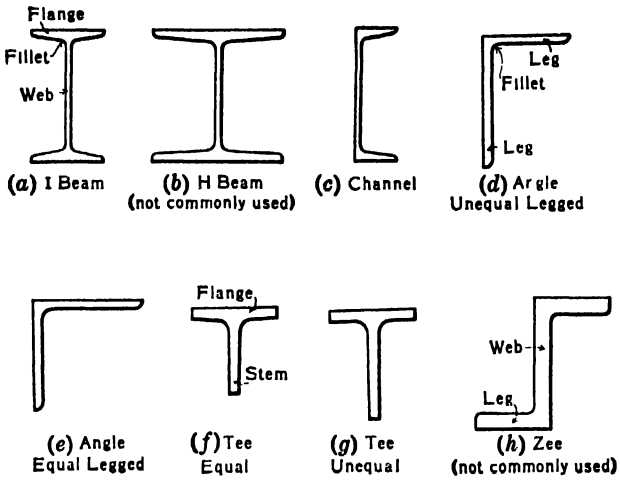


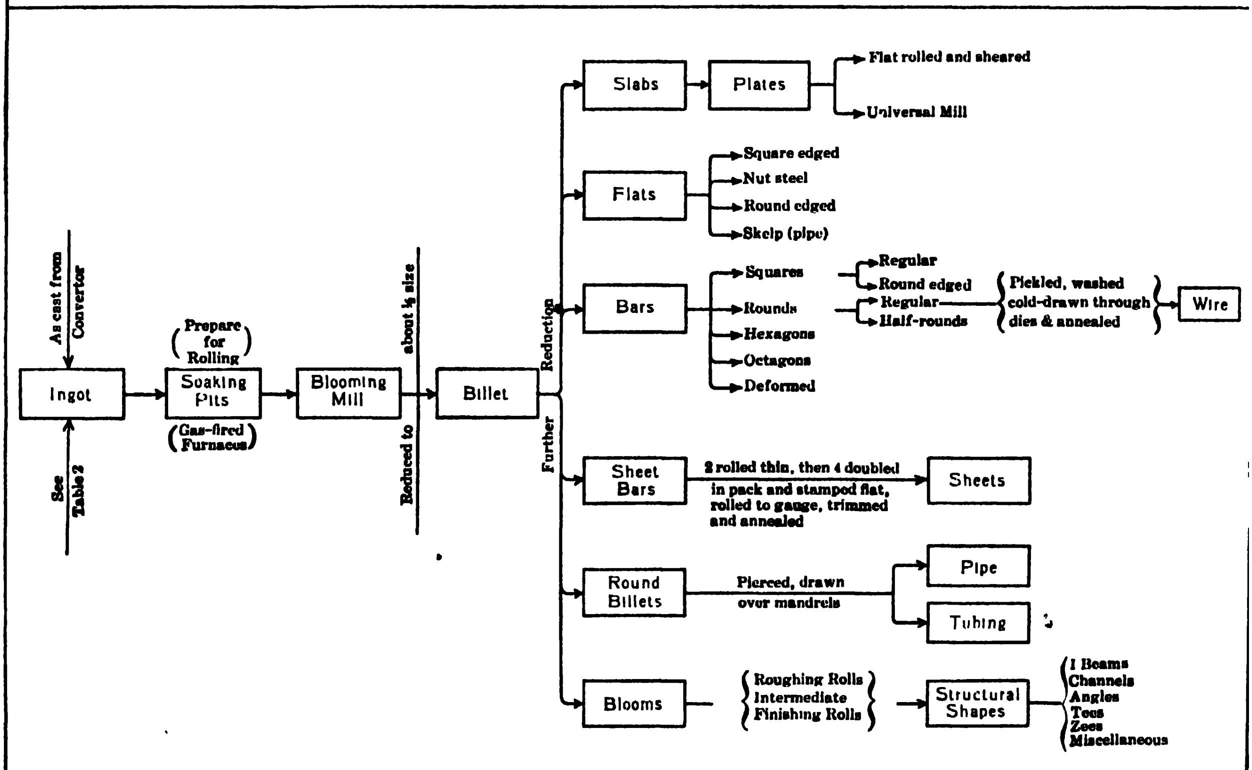
FIG. 3

details, while zees which were once common in column work are now seldom used, as the methods of making up columns have changed.

For structural shapes, there is a **minimum section** upon which other sizes of the same nominal depth are based. The latter are obtained by **spreading the rolls** to increase the section areas and weights, as illustrated in Fig. 4. "The cross-hatched portions represent the minimum sections and the blank portions the added areas. In the case of Channels, I Beams and Bulb Beams, the enlargement of the section adds an equal amount to the thickness of the web and the width of the flanges. In the case of Angles and Zees, the effect of spreading the rolls is slightly to increase the length of the legs. Many of the sizes, however, are rolled in finishing passes, whereby the exact dimensions are maintained for different thicknesses. Inasmuch, however, as these passes are modified in the wear of the rolls, it is impracticable to state what the exact dimensions will be, except in the case of the minimum weight sections. Designers and detailers of structural work should, therefore, arrange for ample clearances."* Table 3 summarizes the main steps in reducing the ingots to commercial forms.

TABLE 3

ROLLING OF SHAPES



3. I Beams.

I beams, the most commonly used of the structural shapes, are classed as **standard** and **Bethlehem**:

* See Carnegie Pocket Companion.

The so-called standard beams are those established by the Association of American Steel Manufacturers and they are rolled by several mills. The use of the word standard here does not mean " accepted " in the sense that the other beams are not satisfactory. Bethlehem beams are so named as they are a product of the Bethlehem Steel Company. They are rolled in a different type of mill which is

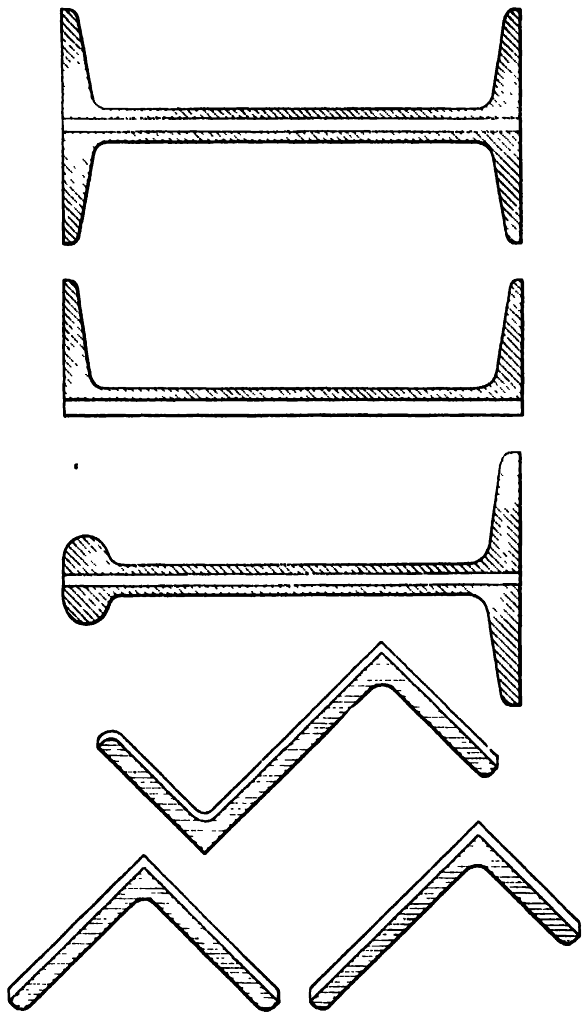


Fig. 4

called the Grey Universal. In a standard beam mill, the rolls form the web principally, while the ends of the rolls simply press against the flanges, with the outsides of the flanges acting as stationary guides. Since there is a limit to forcing the metal out without actually rolling at such points, the material may not be as dense there as that in the web. The Grey Universal mill has both horizontal and vertical rolls, as indicated in Fig. 5 (*a*), which form the web and flanges coincidentally. It is claimed that this uniform reduction of the ingot eliminates the condition of internal stress caused by unequal deformations in rolling by the ordinary methods. Figure 5 (*d*) shows the method of spreading the rolls to increase a section to one larger than the minimum.

Each of the two types of beams has its advantages according to the requirements of each particular case. By contrast, any particular Bethlehem section has a wider flange, and since its weight is about

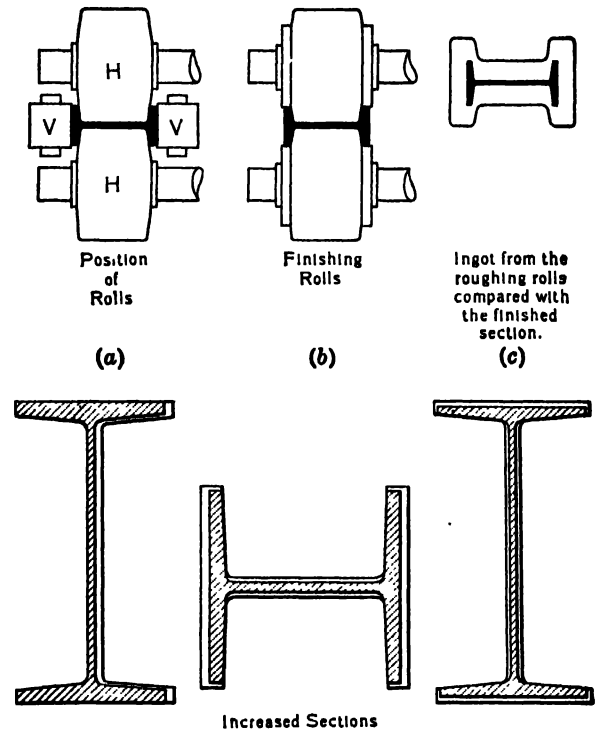


Fig. 5

the same as the standard section which is comparable with it, it has a thinner web. The wide flange is convenient when it is desired to carry a wall or column, whereas the standard section, with its narrower flange, is easier to conceal in partitions if necessary. Since the web of the Bethlehem section is thinner, shear and buckling stresses may more often control here, but wide flanges require less lateral support. The Bethlehem section also distributes a greater portion of its steel where it will be of most flexural value.

Figure 6 (*a*) shows the established proportions of a typical **standard section,** and (*b*) the scale for the minimum sections. It should be noted that the depths range from 3″ to 24″ inclusive. The 6″ depth is usually the smallest employed for a structural beam, and in heavy framing, the 8″ beam is often made a minimum. The smaller depths, namely 3″, 4″ and 5″, are usually employed in the fabrication of ornamental iron work and framing for special equipment. A rolled steel beam is de-

scribed by its depth (in inches) and its weight per lineal foot (in pounds), as for example:

12 I 31.8 × 14′-3″,
2-6 Ils $12\frac{1}{4}$ × 6′-8″,
24 I 79.9 × 17′-$9\frac{5}{16}$″, etc.

Table 4 gives the properties of the common standard beams. There are many sizes which are not listed that are rolled and their properties may be found in the structural handbooks. As a rule, the minimum weights of a given depth are the most commonly found in stock in the average structural companies, and they are also the easiest to obtain from the mills, since they are the product of more frequent rollings. It is unwise, therefore, for the designer to call for an "odd" beam when the chances are that it will be substituted for in the subsequent steps of planning the frame. The properties of the sections are established by using the methods established by the fundamental principles of mechanics relating to moment of inertia, section modulus, and so on.

Illustrative Prob. 3a. Calculate the elements of a 24 I 79.9 about the 1–1 axis (see Fig. 7).

Area

$$\left[7 \times 0.6 + \frac{0.54 \times 3.25}{2} \times 2 + 11.4 \times 0.5\right] \times 2 = 23.32\square''.$$

Moment of Inertia $I = I_0 + A \cdot d^2$

$$\text{Flange rectangle } \frac{b \cdot d^3}{12} = \frac{7 \times (0.60)^3}{12} = 0.22$$

$$A \cdot d^2 = (7 \times 0.60) \times (11.7)^2 = 575.$$

$$\text{Flange triangles } \frac{b \cdot h^3}{36} = \frac{3.25 \times (0.54)^3}{36} \times 2 = 0.03$$

$$A \cdot d^2 = \left(\frac{3.25 \times 0.54}{2}\right) \times 2 \times (11.22)^2 = 221.$$

$$796.25$$
$$2$$
$$1592.5$$

$$\text{Web rectangle } \frac{b \cdot d^3}{12} = \frac{0.5 \times (22.8)^3}{12} = 494.7$$

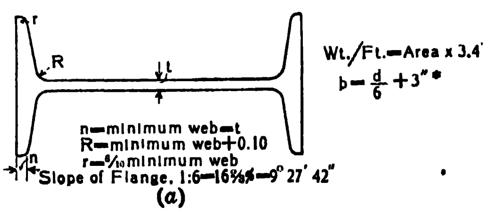
$$I(1\text{–}1) = 2087.2''$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{2087.2}{23.32}} = 9.46'' \quad c = 12, \quad \frac{I}{c} = \frac{2087.2}{12} = 173.9''^3.$$

TABLE 4

ELEMENTS OF COMMON STANDARD BEAMS

Depth of Beam	Wt. per Foot	Area of Section	Width of Flange	Thickness of Web	Axis 1-1			Axis 2-2		
					I	r	$\frac{I}{c}$	I	r	$\frac{I}{c}$
In.	Lbs.	In.^{2}	In.	In.	In.^{4}	In.	In.^{3}	In.^{4}	In.	In.^{3}
24	**105.0**	30.98	7.875	0.625	2811.5	9.53	**234.3**	78.9	1.60	20.0
	100.0	29.41	7.254	0.754	2379.6	9.00	**198.3**	48.6	1.28	13.4
	90.0	26.47	7.131	0.631	2238.4	9.20	**186.5**	45.7	1.31	12.8
	79.9	23.32	7.000	0.500	2087.2	9.46	**173.9**	42.9	1.36	12.3
20	**80.0**	23.73	7.000	0.600	1466.3	7.86	**146.6**	45.8	1.39	13.1
	65.4	19.08	6.250	0.500	1169.5	7.83	**117.0**	27.9	1.24	8.9
18	**75.0**	22.05	7.000	0.562	1141.3	7.19	**126.8**	46.2	1.45	13.2
	54.7	15.93	6.000	0.460	795.6	7.07	**88.4**	21.2	1.15	7.1
15	**60.0**	17.67	6.000	0.590	609.0	5.87	**81.2**	26.0	1.21	8.7
	42.9	12.48	5.500	0.410	441.8	5.95	**58.9**	14.6	1.08	5.3
12	**40.8**	11.84	5.250	0.460	269.0	4.77	**44.8**	13.8	1.08	5.3
	31.8	9.26	5.000	0.350	215.8	4.83	**36.0**	9.5	1.01	3.8
10	**25.4**	7.37	4.660	0.310	122.1	4.07	**24.4**	6.9	0.97	3.0
9	**21.8**	6.31	4.330	0.290	84.9	3.67	**18.9**	5.2	0.90	2.4
8	**18.4**	5.33	4.000	0.270	56.9	3.27	**14.2**	3.8	0.84	1.9
7	**15.3**	4.42	3.660	0.250	36.2	2.86	**10.4**	2.7	0.78	1.5
6	**12.5**	3.61	3.330	0.230	21.8	2.46	**7.3**	1.9	0.72	1.1



(a)

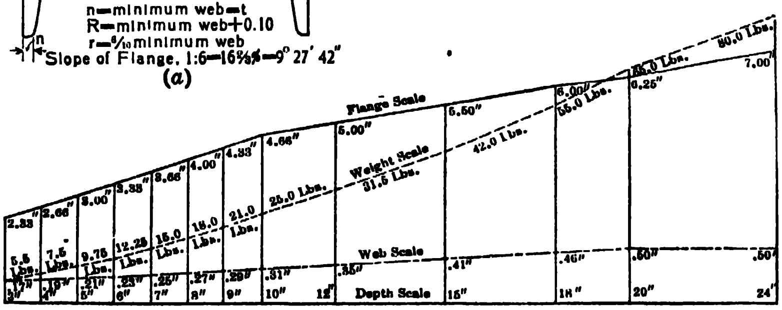


DIAGRAM FOR MINIMUM STANDARD BEAMS

(b)

FIG. 6

* This relation is one which is convenient to establish a flange width when no handbook is available. Thus for a 15 I, $b = \frac{15}{6} + 3 = 5.5''$.

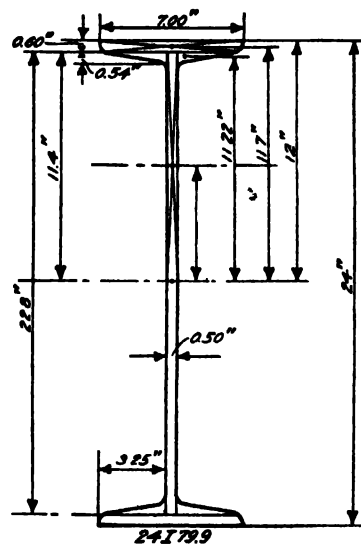


FIG. 7

The majority of the new editions of structural handbooks list the minimum weights of beams to the nearest 0.1 of a pound, replacing the well established custom of giving the weight to the nearest one-half pound. The reason for so doing is said to be that the old weights, as were given, never had shown the true weight of the beam, and, in estimates, the difference might make an appreciable error for a large amount of steel, and more particularly, the price of the beam would be in error. The authors agree that these new weights should be well known to estimators, but they do not see the need for calling for the beam's exact weight in all instances, such as on structural plans. In timber work, plans may call for a 6″ × 10″, when the designer knows full well that he will obtain a beam smaller than these nominal dimensions and accordingly, he makes allowances for this variation. The use of what may be termed nominal weights for steel work has heretofore been satisfactory and the indication of a beam as a 12 I 31½ would call for a weight of 31.8# when the plan is estimated. The properties have not been changed by what seems to be an increase in weight, which evidently proves that the 12 I 31½ always weighed 31.8# or thereabouts. The authors also believe that the weights for the oversize beams are similarly in error but these have not been changed, inasmuch as they are not commonly used. It is a well known fact that beams will vary in weight at the start of a rolling from the weight at the close of the rolling, as new rolls cause a slight underweight and old rolls an overweight. Moreover, steel is sold at the mill by scale weight and is estimated for contract purposes by the weight per foot.

TABLE 5

PROPERTIES OF COMMON BETHLEHEM GIRDER BEAMS

Nominal Depth of Beam, In.	Weight, Lbs. per Foot	Area of Section, Sq. In.	Thickness of Web. In.	Width of Flange In	Axis 1-1 Moment of Inertia I	Axis 1-1 Radius of Gyration r	Axis 1-1 Section Modulus $\frac{I}{c}$	Axis 2-2 Moment of Inertia I	Axis 2-2 Radius of Gyration r	Axis 2-2 Section Modulus $\frac{I}{c}$
30	**200.0**	58.52	76	15 04	9148 8	12 50	**607.5**	628.5	3.28	83.6
	181.0	52 82	69	14 97	8181 0	12 45	**547.6**	552 0	3 23	73 7
28	**165.0**	48 19	.66	14 25	6577.9	11.68	**469.9**	462 8	3.10	65.0
26	**151.0**	44 16	63	13 75	5237 1	10 89	**402.9**	402 7	3 02	58 6
24	**141.0**	41 02	61	13 25	4174 2	10 09	**347.9**	356 4	2 95	53 8
	121.0	35.30	.54	12.25	3585 3	10.08	**298.8**	256.9	2.70	41.9
20	**142.0**	41 31	66	12 75	2932 3	8 43	**293.2**	360 9	2 96	56 6
	113.0	32.90	56	12 00	2340 2	8 43	**234.0**	240 8	2 71	40 1
18	**93.0**	27.14	48	11.50	1593 4	7 66	**177.0**	185.1	2 61	32 2
15	**141.0**	40 86	80	11 75	1577 7	6 21	**210.4**	328 3	2 83	55 9
	105.0	30 45	60	11.25	1218.2	6.32	**162.4**	214 3	2 65	38 1
	74.0	21 55	.44	10 75	883.8	6 40	**117.6**	128.9	2 45	24 0
12	**70.5**	20 57	.47	10 25	538 4	5.12	**89.7**	119 7	2.41	23 4
	55.5	16.21	.38	10.00	431 8	5.16	**72.0**	84 9	2 29	17.0

Conforming to revised catalogue of Oct. 1, 1922.
For other sizes, see catalogue of Bethlehem Steel Co.

The maximum lengths of beams obtainable is dependent upon the particular mill, but they average

75′-0″ for 24″ to 12″ I's,
70′-0″ for 10″ to 5″ I's, and
50′-0″ for 4″ and 3″ I's.

Bethlehem beams are classed in two definite groups, namely, Bethlehem **I Beams,** and Bethlehem **Girder Beams.** The former are rolled in depths from 8″ to 30″, and the latter in the same depths but with thicker metal. Girder beams can be used when plate girders or other built-up sections might otherwise be necessary, and hence considerable fabrication may be eliminated. In order to discriminate Bethlehem beams from standard beams, and also to discriminate the Bethlehem beams from the Bethlehem girder beams, the following practice is generally followed:

12 BI 36 × 14′-0″,
20 BI 59 × 17′-0″,
26 BG 150 × 22′-8¾″,
30 BG 200 × 47′-6″, etc.

TABLE 6

PROPERTIES OF COMMON BETHLEHEM I BEAMS

Depth of Beam, In.	Weight, Lbs. per Foot	Area of Section, Sq. In.	Thickness of Web. In.	Width of Flange, In.	Axis 1-1 Moment of Inertia I	Axis 1-1 Radius of Gyration r	Axis 1-1 Section Modulus $\frac{I}{c}$	Axis 2-2 Moment of Inertia I	Axis 2-2 Radius of Gyration r	Axis 2-2 Section Modulus $\frac{I}{c}$
30	**121.0**	35 30	540	10 500	5239 6	12 18	**349.3**	165 0	2 16	31 4
28	**106.0**	30 88	500	10.000	4014 1	11.40	**286.7**	131 5	2 06	26.3
26	**91.0**	26 49	460	9 500	2977.2	10.60	**229.0**	101.2	1.95	21 3
24	**73.5**	21.47	390	9.000	2091 0	9 87	**174.3**	74.4	1 86	16.5
20	**73.0**	21.37	430	8 750	1466 5	8 28	**146.7**	75.9	1.88	17.3
	59.5	17 36	.375	8.000	1172 2	8 22	**117.2**	48 3	1 66	12 1
18	**49.0**	14 25	320	7 500	708 3	7 48	**88.7**	36.2	1 59	9 66
15	**54.5**	15.88	410	7.000	610.0	6.20	**81.3**	38.3	1.55	10 9
	38.5	11.27	290	6 660	442.6	6.27	**59.0**	23 4	1.44	7 03
12	**32.0**	9.44	.335	6.205	228.5	4.92	**38.1**	16.0	1 30	5 14
	28.5	8.42	.250	6.120	216.2	5 07	**36.0**	15 3	1.35	4.98
10	**28.5**	8 34	.390	5 990	134 6	4.02	**26.9**	12.1	1 21	4.05
	23.5	6 94	250	5 850	122.9	4.21	**24.6**	11.2	1.27	3.83

Conforming to revised catalogue of Oct. 1, 1922.
For other sizes, see catalogue of Bethlehem Steel Co.

Table 5 gives the properties of common Bethlehem I beams and Table 6 those of Bethlehem girder beams which are usually carried in stock. The 8″, 9″ and 10″ girder beams and the 8″ and 9″ beams are not commonly used, and standard sections are generally employed for such members instead.

Prob. 3b. Calculate the elements of a 24 I 79.9 about the 2-2 axis. See Fig. 7. Check against tables.
Prob. 3c. Calculate the elements of a 30 BG 200 about the 1-1 axis. Refer to handbook for dimensions. Check against table.

4. Structural Channels.

Structural channels are similar to I beams except that the flange projects to one side only. Figure 8 (*a*) gives the standard proportions of a typical section, and (*b*) the scale of dimensions. There has been some discussion* which favors the making of the ship-building and the structural channels the same standard sections, but at present this is not in effect, and the **ship-building channel should not be confused with the structural channel.** The channel is identified by its depth (in inches) and the weight per lineal foot (in pounds) as for example,

15 ⊏ 33 × 13′-0$\frac{1}{16}$″,
9 ⊏ 13$\frac{1}{4}$ × 17′-10$\frac{1}{4}$″, etc.

Table 7 gives the properties of the more usual stock of channels, which range in depth from 3″ to 15″, although the 3″, 4″ and 5″ channels are used

TABLE 7†

ELEMENTS OF COMMON STRUCTURAL CHANNELS

Depth of Beam	Weight per Foot	Area of Section	Width of Flange	Thickness of Web	Axis 1-1			Axis 2-2			
					I	r	$\frac{I}{c}$	I	r	$\frac{I}{c}$	ig
In.	Lbs.	In.^{2}	In.	In.	In.^{4}	In.	In.^{3}	In.^{4}	In.	In.^{3}	In.^{4}
15	**40.0**	11 70	3.52	0 52	346.3	5 44	**46.2**	9 3	0 89	3 4	0 78
	33.9	9.00	3 40	0 40	312 6	5 62	**41.7**	8 2	0 91	3 2	0 79
12	**30.0**	8.70	3.17	0 51	161.2	4 28	**26.9**	5 2	0 77	2 1	0 68
	20.7	6.03	2 94	0 28	128 1	4 61	**21.4**	3.9	0 81	1 7	0 70
10	**25.0**	5 86	2.74	0 38	78 5	3 66	**15.7**	2 8	0 70	1 3	0 61
	15.3	4 47	2.60	0.24	66.9	3 87	**13.4**	2.3	0 72	1 2	0 64
9	**13.4**	3 89	2.43	0 23	47 3	3 40	**10.5**	1 8	0 67	0 9	0 61
8	**11.5**	3 36	2.26	0 22	32 3	3 10	**8.1**	1 3	0 63	0 79	0 58
7	**9.8**	2 85	2 09	0 21	21 1	2 72	**6.0**	0 9	0 50	0 63	0 55
6	**8.2**	2.39	1.92	0.20	13 0	2 34	**4.3**	0.7	0 54	0 50	0 52

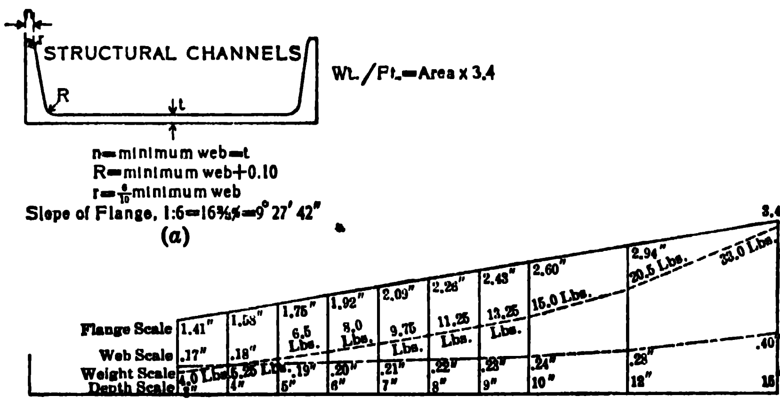


DIAGRAM FOR MINIMUM STANDARD CHANNELS
(*b*)

FIG. 8

The shape may be listed on bills and indicated on plans so that its final position in the structure is evident. For example:

flanges up, web horizontal ⊔,
flanges facing left ⊐,
flanges facing right ⊏.

in ornamental iron work principally. For the properties of the sizes not shown in Table 7, any structural handbook may be consulted. The maximum lengths of channels rolled average

75′-0″ for 15″ and 12″ depths,
70′-0″ for 10″ to 5″ depths, and
50′-0″ for 4″ and 3″ depths.

The properties may be calculated in a manner similar to that used in the case of the I beams.

* "Proposals Formulated by the Sectional Committee on Steel Shapes under the Auspices of the American Standards Committee for Submission to its Sponsor Organizations, to the American Engineering Standards Committee and to the British Engineering Standards Association as a Basis for Common Anglo-American Standards." Proc. A.S.C.E., August, 1920, and Pamphlet B.E.S.A., C.L. 7777, London, Nov., 1918.

† For other sizes, see "Pocket Companion," Carnegie Steel Co.

Prob. 4a. Calculate the value of $I_{1\text{-}1}$ for a 15 [33.9. Refer to structural tables (Carnegie Pocket Companion) for dimensions.

5. Structural Angles.

Structural angles are rolled in two distinct groups, namely, equal legged, and unequal legged, there being eight base sizes for the former and nine for the latter. An angle is described by giving the dimensions of the two legs and the thickness of the metal as

$$3 \times 3 \times \tfrac{1}{4}\ \text{L} \times 4'\text{-}0'',$$
$$6 \times 3\tfrac{1}{2} \times \tfrac{3}{8}\ \text{L} \times 12'\text{-}0'', \text{ etc.}$$

In the case of unequal legged angles, the longer leg is always written first. The base sizes are increased by the rolls as shown in Fig. 4, and Fig. 9 shows the standard proportions. In the method of rolling, angles (except minimum sizes) are subject to **over-run** unless they are passed through finish-

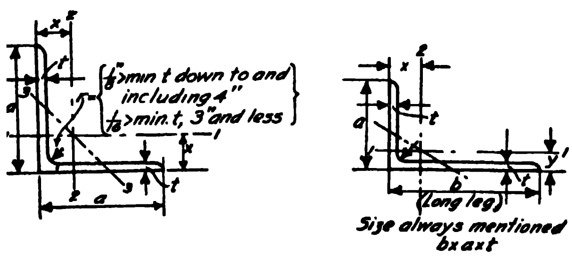


FIG. 9

ing rolls, which is not common for the majority of sizes. The over-run in each leg is equal to the amount the rolls are opened above the minimum requirement, except for slight variations such as those caused by the wearing of the rolls. For the three angles listed below, the over-run is as follows:

Nominal Size	$3\times3\times\frac{1}{4}$ (Min.)	$3\times3\times\frac{5}{16}$	$3\times3\times\frac{3}{8}$
Over-run Size (Actual)	$3\times3\times\frac{1}{4}$	$3\frac{1}{16}\times3\frac{1}{16}\times\frac{5}{16}$	$3\frac{1}{8}\times3\frac{1}{8}\times\frac{3}{8}$

The maximum commercial lengths of angles available vary from 60′-0″ to 80′-0″. Tables 8 and 9 give the properties of the common sizes of angles employed. The properties of other sizes may be found in the usual structural handbooks. They are calculated by employing the same general principles as described for the I beams.

Prob. 5a. Calculate the elements of a $6 \times 6 \times \frac{3}{8}$ angle.

6. Structural Tees and Zees.

Structural tees are rolled in two groups similar to angles, namely equal and unequal, such as shown in Fig. 10 (a) and (b). Special tees are also rolled but these are used for particular instances. Because of the varying thickness of the metal in any given section, the sizes are defined by

Flange width × depth × weight per foot,

as a 4 × 3 × 9.2# T, and a symbol, T or ⊥, indicates the position of the member in place. Table 10 gives only the properties of tees commonly used. These are generally limited to a very few small sizes in practice.*

Figure 10 (c) shows a typical zee. These are very seldom used in present practice. Table 11

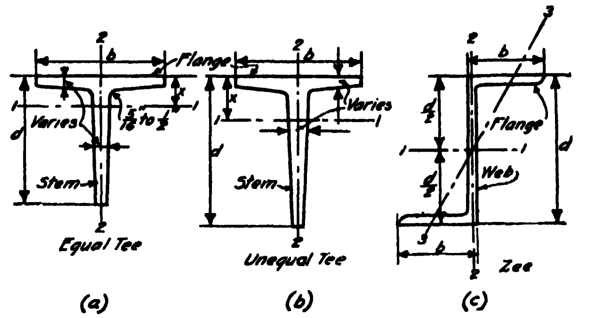


FIG. 10

TABLE 8*

ELEMENTS OF COMMON EQUAL ANGLES

Size		Weight per Foot	Area of Section	Axis 1-1 and Axis 2-2				Axis 3-3
				I	r	$\frac{I}{c}$	x	r min.
In.		Lbs.	In.²	In.⁴	In.	In.³	In.	In.
6 × 6	$\frac{7}{8}$	33 1	9 73	31 9	1 81	7 6	1 82	1 17
	$\frac{3}{4}$	28 7	8 44	28 2	1 83	6 7	1 78	1 17
	$\frac{5}{8}$	24 2	7 11	24 2	1 84	5 7	1 73	1 17
	$\frac{1}{2}$	19 6	5 75	19 9	1 86	4 6	1 68	1 18
	$\frac{3}{8}$	14 9	4 36	15 4	1 88	3 5	1 64	1 19
4 × 4	$\frac{1}{2}$	12 8	3 75	5 6	1 22	2 0	1 18	0 78
	$\frac{3}{8}$	9 8	2 86	4 4	1 23	1 5	1 14	0 79
	$\frac{5}{16}$	8 2	2 40	3 7	1 24	1 3	1 12	0 79
$3\frac{1}{2} \times 3\frac{1}{2}$	$\frac{1}{2}$	11 1	3 25	3 6	1 06	1.5	1 06	0 68
	$\frac{3}{8}$	8 5	2 48	2 9	1 07	1 2	1 01	0 69
	$\frac{5}{16}$	7 2	2.09	2 5	1 08	0 98	0 99	0.69
	$\frac{1}{4}$	5.8	1 69	2 0	1 09	0 79	0 97	0 69
3 × 3	$\frac{3}{8}$	7 2	2 11	1 8	0.91	0.83	0 89	0 58
	$\frac{5}{16}$	6.1	1.78	1 5	0.92	0 71	0 87	0 59
	$\frac{1}{4}$	4 9	1.44	1 2	0.93	0 58	0 84	0 59
$2\frac{1}{2} \times 2\frac{1}{2}$	$\frac{3}{8}$	5 9	1 73	0 98	0.75	0 57	0 76	0 48
	$\frac{5}{16}$	5 0	1.47	0 85	0.76	0 48	0 74	0.49
	$\frac{1}{4}$	4 1	1 19	0.70	0 77	0 39	0.72	0.49

* For other data, see Carnegie Pocket Companion.

TABLE 9*

ELEMENTS OF COMMON UNEQUAL ANGLES

Size		Wt. per Foot	Area of Section	Axis 1-1				Axis 2-2				Axis 3-3
				I	r	$\frac{I}{c}$	x	I	r	$\frac{I}{c}$	y	r min.
In.		Lbs.	In.²	In.⁴	In.	In.³	In.	In.⁴	In.	In.³	In.	In.
6 × 4	7/8	27.2	7.98	27.7	1.86	7.2	2.12	9.8	1.11	3.4	1.12	0.86
	3/4	23.6	6.94	24.5	1.88	6.2	2.08	8.7	1.12	3.0	1.08	0.86
	5/8	20.0	5.86	21.1	1.90	5.3	2.03	7.5	1.13	2.5	1.03	0.86
	1/2	16.2	4.75	17.4	1.91	4.3	1.99	6.3	1.15	2.1	0.99	0.87
	3/8	12.3	3.61	13.5	1.93	3.3	1.94	4.9	1.17	1.6	0.94	0.88
6 × 3½	3/4	22.4	6.56	23.3	1.89	6.1	2.18	5.8	0.94	2.3	0.93	0.75
	5/8	18.9	5.55	20.1	1.90	5.2	2.13	5.1	0.96	1.9	0.88	0.75
	1/2	15.3	4.50	16.6	1.92	4.2	2.08	4.3	0.97	1.6	0.83	0.76
	3/8	11.7	3.42	12.9	1.94	3.3	2.04	3.3	0.99	1.2	0.79	0.77
5 × 3½	5/8	16.8	4.92	12.0	1.56	3.7	1.70	4.8	0.99	1.9	0.95	0.75
	1/2	13.6	4.00	10.0	1.58	3.0	1.66	4.0	1.01	1.6	0.91	0.75
	3/8	10.4	3.05	7.8	1.60	2.3	1.61	3.2	1.02	1.2	0.86	0.76
	5/16	8.7	2.56	6.6	1.61	1.9	1.59	2.7	1.03	1.0	0.84	0.76
4 × 3	3/8	8.5	2.48	4.0	1.26	1.5	1.28	1.9	0.88	0.87	0.78	0.64
	5/16	7.2	2.09	3.4	1.27	1.2	1.26	1.7	0.89	0.74	0.76	0.65
	1/4	5.8	1.69	2.8	1.28	1.0	1.24	1.4	0.89	0.60	0.74	0.65
3½ × 3	3/8	7.9	2.30	2.7	1.09	1.1	1.08	1.8	0.90	0.85	0.83	0.62
	5/16	6.6	1.93	2.3	1.10	0.96	1.06	1.6	0.90	0.72	0.81	0.63
	1/4	5.4	1.56	1.9	1.11	0.78	1.04	1.3	0.91	0.58	0.79	0.63
3½ × 2½	3/8	7.2	2.11	2.6	1.10	1.1	1.16	1.1	0.72	0.59	0.66	0.54
	5/16	6.1	1.78	2.2	1.11	0.93	1.14	0.94	0.73	0.50	0.64	0.54
	1/4	4.9	1.44	1.8	1.12	0.75	1.11	0.78	0.74	0.41	0.61	0.54
3 × 2½	3/8	6.6	1.92	1.7	0.93	0.81	0.96	1.0	0.74	0.58	0.71	0.52
	5/16	5.6	1.62	1.4	0.94	0.69	0.93	0.90	0.74	0.49	0.68	0.53
	1/4	4.5	1.31	1.2	0.95	0.56	0.91	0.74	0.75	0.40	0.66	0.53
2½ × 2	3/8	5.3	1.55	0.91	0.77	0.55	0.83	0.51	0.58	0.36	0.58	0.42
	5/16	4.5	1.31	0.79	0.78	0.47	0.81	0.45	0.58	0.31	0.56	0.42
	1/4	3.62	1.06	0.65	0.78	0.38	0.79	0.37	0.59	0.25	0.54	0.42

gives the properties of only a few of the more common sizes.* A zee is described by its nominal depth (in inches) and its weight per lineal foot (in pounds) as for example, a 6 Z 22.8 × 14′-6″.

* For other data and sizes see Carnegie Pocket Companion.

† The sizes so indicated have now (1926) become obsolete and the rolling of these sections has been discontinued.

TABLE 10*

ELEMENTS OF COMMON EQUAL TEES

Size				Wt. per Foot	Area of Section	Axis 1-1				Axis 2-2		
Flanges	Stem	Minimum Thickness				I	r	$\frac{I}{c}$	x	I	r	$\frac{I}{c}$
		Flanges	Stem									
In.	In.	In.	In.	Lbs.	In.²	In.⁴	In.	In.³	In.	In.⁴	In.	In.³
4	4	1/2	1/2	13.5	3.97	5.7	1.20	2.0	1.18	2.8	0.84	1.4
4	4	3/8	3/8	10.5	3.09	4.5	1.21	1.6	1.13	2.1	0.83	1.1
3½†	3½	1/2	1/2	11.7	3.44	3.7	1.04	1.5	1.05	1.9	0.74	1.1
3½†	3½	3/8	3/8	9.2	2.68	3.0	1.05	1.2	1.01	1.4	0.73	0.81
3†	3	1/2	1/2	9.9	2.91	2.3	0.88	1.1	0.93	1.2	0.64	0.80
3†	3	7/16	7/16	8.9	2.59	2.1	0.89	0.98	0.91	1.0	0.63	0.70
3	3	3/8	3/8	7.8	2.27	1.8	0.90	0.86	0.88	0.90	0.63	0.60
3	3	5/16	5/16	6.7	1.95	1.6	0.90	0.74	0.86	0.75	0.62	0.50
2½	2½	3/8	3/8	6.4	1.87	1.0	0.74	0.50	0.76	0.52	0.53	0.42
2½	2½	5/16	5/16	5.5	1.60	0.88	0.74	0.50	0.74	0.44	0.52	0.35

ELEMENTS OF COMMON UNEQUAL TEES

Size				Wt. per Foot	Area of Section	Axis 1-1				Axis 2-2		
Flanges	Stem	Minimum Thickness				I	r	$\frac{I}{c}$	x	I	r	$\frac{I}{c}$
		Flanges	Stem									
In.	In.	In.	In.	Lbs.	In.²	In.⁴	In.	In.³	In.	In.⁴	In.	In.³
5	3	3/8	11/32	11.5	3.37	2.4	0.84	1.1	0.76	3.9	1.10	1.6
4	5	1/2	1/2	15.3	4.50	10.8	1.55	3.1	1.56	2.8	0.79	1.4
4	5	3/8	3/8	11.9	3.40	8.5	1.56	2.4	1.51	2.1	0.78	1.1
4	3	3/8	3/8	9.2	2.68	2.0	0.86	0.90	0.78	2.1	0.89	1.1
4	3	5/16	5/16	7.8	2.29	1.7	0.87	0.77	0.75	1.8	0.88	0.88
3†	4	1/2	1/2	11.7	3.44	5.2	1.23	1.9	1.32	1.2	0.59	0.81
3†	4	3/8	3/8	9.2	2.68	4.1	1.24	1.5	1.27	0.90	0.58	0.60
3†	2½	3/8	3/8	7.1	2.07	1.1	0.72	0.60	0.71	0.89	0.66	0.50
3	2½	5/16	5/16	6.1	1.77	0.94	0.73	0.52	0.68	0.75	0.65	0.50

7. Plates.

Since steel plates are used for many purposes in fabrication work, such as for plate girders, columns and trusses, the sizes vary between wide limits. A point of distinction in the matter of thickness is made by naming material from $\frac{1}{100}''$ up to $\frac{1}{8}''$. **sheets,** and material $\frac{1}{8}''$ thick and above, **plates.** Narrow widths of material are often called **bars**

the limiting maximum width being 4″ or 6″, depending upon local practice. Plates in general are either "**flat rolled and sheared**" or rolled in a **Universal**

TABLE 11
ELEMENTS OF COMMON ZEES

Size			Wt. per Foot	Area of Section	Axis 1-1			Axis 2-2			Axis 3-3
Depth	Flanges	Thickness			I	r	$\frac{I}{c}$	I	r	$\frac{I}{c}$	r min.
In.	In.	In.	Lbs	In.²	In.⁴	In.	In.³	In.⁴	In.	In.³	In.
6	3½	¾	29.4	8.63	42.1	2.21	14.0	15.4	1.34	4.9	0.81
6	3½	9/16	22.8	6.68	34.6	2.28	11.5	12.6	1.37	3.9	0.81
6	3½	⅜	15.7	4.59	25.3	2.35	8.4	9.1	1.41	2.8	0.83
5	3¼	11/16	23.7	6.96	23.7	1.84	9.5	11.4	1.28	3.9	0.73
5	3¼	½	17.9	5.25	19.2	1.91	7.7	9.1	1.31	3.0	0.74
5	3¼	5/16	11.6	3.40	13.4	1.98	5.3	6.2	1.35	2.0	0.75
4	3 1/16	⅝	18.9	5.55	12.1	1.48	6.1	8.7	1.25	3.2	0.66
4	3 1/16	7/16	13.8	4.05	9.7	1.55	4.8	6.7	1.29	2.4	0.66
4	3 1/16	¼	8.2	2.41	6.3	1.62	3.1	4.2	1.33	1.4	0.67
3	2 11/16	½	12.6	3.69	4.6	1.12	3.1	4.9	1.15	2.0	0.53
3	2 11/16	⅜	9.8	2.86	3.9	1.16	2.6	3.9	1.17	1.6	0.54
3	2 11/16	¼	6.7	1.97	2.9	1.21	1.9	2.8	1.19	1.1	0.55

For other sizes, see "Pocket Companion," Carnegie Steel Co.

Mill (Art. 2).* In the former, the plates must be straightened after rolling and the uneven edges sheared off, while in the latter, the edges are rolled to uniform dimensions. By the nature of the latter process, Universal Mill (U. M.) plates are limited to thicknesses of ¼″ and greater, and to widths of 48″ and less. Plates may be sheared to any width desired, but the common sizes which may be obtained as Universal Mill material are limited to

widths < 4″ — by ½″ increments
widths > 4″ — by 2″ increments up to 48″.

The usual sizes of "flat rolled and sheared" plates, although varying with the particular mill and the demand, are as follows:

Widths	Thicknesses
1½, 1¾, 2	3/16 and ¼†
2¼, 2½, 3	
3½, 4″ to 10″ by 1″ increments	¼″ to 2½″ maximum
12″ up, by 2″ increments to 132″, with variations of ±¼″	

* Plates may also be bought in circular shapes. For checkered plates, see Index. For a full discussion of such products, see Part XII, "Building Materials," Vol. I.

† In very heavy work, plates are sometimes designated by their weight per sq. ft., as plates are slightly thicker in the middle than at the edges, due to the springing of the rolls.

Table 12 gives the maximum lengths obtainable. The variation in lengths is ±⅜″. Although plates'

TABLE 12
DATA FOR STEEL PLATES

RECTANGULAR UNIVERSAL PLATES — Carbon Steel Universal Mill Plates, One-Fourth Inch and Over, Extreme Sizes

Thickness, Inches	Weight, Lbs. per Sq. Ft.	Widths and Lengths in Inches										
		48–46	45–41	40–36	35–31	30–26	25–20	19–17	16–15	14–12	11	10–6½
¼	10.20						1020	1020	1020	1020	540	540
5/16	12.75	1020	1020	1140	1260	1320	1320	1080	1080	1080	600	600
⅜	15.30	1200	1200	1320	1380	1380	1380	1080	1080	1080	900	840
7/16	17.85	1320	1320	1380	1380	1380	1380	1080	1080	1080	900	840
½	20.40	1380	1380	1380	1380	1380	1380	1080	1080	1080	1020	840
9/16	22.95	1380	1380	1380	1380	1380	1380	1080	1080	1080	1020	840
⅝	25.50	1380	1380	1380	1380	1380	1380	1080	1080	1080	1020	840
¾	30.60	1353	1357	1363	1372	1380	1380	1080	1080	1080	900	840
⅞	35.70	1160	1163	1169	1177	1188	1203	1080	1080	1080	900	840
1	40.80	1015	1018	1023	1030	1039	1052	1080	1080	1080	900	840
1⅛	45.90	903	905	910	916	924	936	1080	1080	1080	840	840
1¼	51.00	812	814	818	824	832	842	1071	1080	1080	840	840
1⅜	56.10	738	740	744	749	756	766	973	1080	1080	840	840
1½	61.20	677	679	682	687	693	702	892	1059	1080	840	840
1⅝	66.30	625	626	629	634	640	648	823	978	1080	840	840
1¾	71.40	580	581	584	588	594	601	765	908	1038	720	720
1⅞	76.50	541	543	545	549	554	561	714	847	968	660	720
2	81.60	507	509	511	515	519	526	669	794	907	600	720

Plates of greater dimensions than shown in above table may be submitted for special consideration.

RECTANGULAR AND CIRCULAR PLATES — Carbon Steel Sheared Plates, One-Fourth Inch and Over, Extreme Sizes

Thickness, Inches	Weight, Lbs. per Sq. Ft.	Widths and Lengths in Inches										Diam., Inches
		132	126	120	114	108	102	96	90	84	78	
¼	10.20				175	250	280	300	330	375	400	115
5/16	12.75			240	270	320	360	380	420	440	460	120
⅜	15.30	180	240	270	320	365	380	410	450	500	550	132
7/16	17.85	200	270	300	360	370	410	430	460	510	550	132
½	20.40	240	270	320	365	400	450	480	510	550	580	134
9/16	22.95	240	270	330	373	420	470	500	530	570	600	134
⅝	25.50	240	300	350	390	450	500	520	540	600	620	134
11/16	28.05	240	300	360	420	450	500	520	540	600	620	134
¾	30.60	240	300	360	400	450	490	520	540	600	620	134
13/16	33.15	240	300	340	385	440	490	510	530	600	620	134
⅞	35.70	240	300	330	375	440	480	510	530	600	620	134
1	40.80	240	300	300	340	440	460	500	530	580	600	134
1⅛	45.90	240	300	300	330	410	440	450	500	550	580	132
1¼	51.00	230	270	300	310	380	400	420	490	530	550	132
1½	61.20	210	230	260	280	330	320	340	420	440	480	132
1¾	71.40	200	200	220	240	280	270	300	380	380	410	132
2	81.60	180	180	190	210	240	240	260	320	330	360	132
2¼	91.80	132	160	170	190	210	210	230	280	295	320	132
Thickness, Inches	Weight, Lbs. per Sq. Ft.	72	66	60	54	50	48	42	36	30	24	Diam., Inches
¼	10.20	430	475	525	530	530	530	530	530	530	530	115
5/16	12.75	480	500	550	550	575	575	550	550	550	580	120
⅜	15.30	600	600	620	620	620	620	600	580	600	600	132
7/16	17.85	600	630	630	640	640	640	600	580	600	600	132
½	20.40	610	630	630	640	640	640	600	580	630	600	134
9/16	22.95	620	640	640	640	640	640	600	580	630	600	134
⅝	25.50	620	640	640	640	640	640	600	580	600	600	134
11/16	28.05	620	640	640	640	640	640	600	580	600	580	134
¾	30.60	620	640	640	640	640	640	600	580	600	580	134
13/16	33.15	620	640	640	640	640	640	600	580	570	550	134
⅞	35.70	620	640	640	640	640	640	600	580	550	550	134
1	40.80	600	630	630	640	640	640	580	580	520	530	134
1⅛	45.90	580	620	620	640	640	640	580	580	520	500	132
1¼	51.00	550	600	600	600	600	600	560	560	520	450	132
1½	61.20	530	600	600	600	600	600	540	540	470	430	132
1¾	71.40	450	490	550	550	550	550	540	540	430	380	132
2	81.60	400	440	480	500	500	500	500	500	400	350	132
2¼	91.80	350	390	420	450	450	450	450	450	300	200	132

Plates 48″ wide and under may also be rolled on Universal Mills.
For greater length and Universal Mill Sizes, see Universal Mill Plate Table.
Plates of greater dimensions than shown in above tables may be submitted for special consideration.

are obtainable in a number of small thicknesses as suggested above, a minimum thickness of $\frac{1}{4}''$ is established for all structural work for practical reasons, and in many cases, $\frac{5}{16}''$ or even $\frac{3}{8}''$ is specified as the minimum, particularly when the metal is subject to corrosion. A plate is defined usually by its width by its thickness by its length as

2-20 × $\frac{7}{8}$ Pls. × 24′-0″,
1- 4 × $\frac{1}{2}$ bar × 1′-8$\frac{1}{2}$″, etc.

A structural designer in his work should be familiar with costs of structural steel work in general, so that he may be able to estimate the costs in special cases when necessary. The cost of structural steel shapes is referred to **base prices.** Pittsburgh is the recognized center of the steel production and is the basing point for steel prices. The base at any other point is determined by adding the freight rate. The base price varies slightly from month to month, due to manufacturing conditions.

To the cost of the raw material, must be added the costs of making shop details, templates, fabrication, painting, shipment and erection. Structural fabricating companies keep records of these costs so that they are able to arrive at costs of fabricated material erected. These vary from time to time naturally, but a designer may find out from local companies, approximate values for purposes of comparative designs. Such values will obviously depend upon whether the steel is simple beams, plate girders, truss work, or other complicated framing. A value for average work may be established, however. At present (1926), average structural steel costs about $100 per ton in place.

CHAPTER 2

SIMPLE BEAMS

8. Typical Design Example.

In order to illustrate the usual points which are investigated in the design of a typical steel beam,

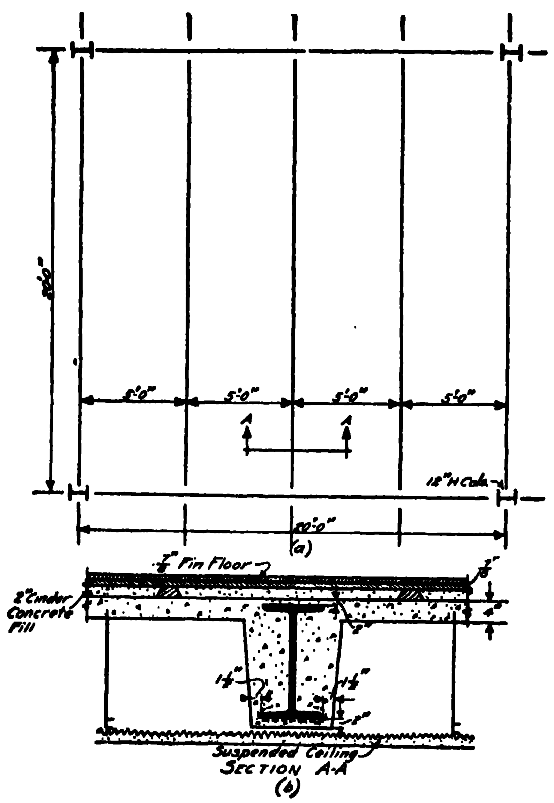


FIG. 11

the floor beam in Fig. 11 will be designed for the following data:

Span = 20'-0" (c.c. bearings as is usual for steel beams)

L. L. =	60#/□'	
Fin. Flr. =	3	s = 16,000#/□"
Sub. Flr. =	3	q = 10,000#/□"
2" Cinder Fill =	16	Maximum allowable deflection $\frac{1}{360}$ of the span.
4" Concrete Slab =	50	Beam is laterally supported by the floor slab.
Suspended Ceil. =	15	
Floor T. L. =	147	
Random Partitions =	13	
T. L. =	160#/□'	

The design work is carried along as follows: Load in #/ft. from the floor = 5 × 160 = 800#/ft. Assume 12" beam. Haunch is then 8" × 12" below the slab (Fig. 11, section A–A).

$$\frac{8 \times 12}{144} \times 150 = 100\#/\text{ft. for beam and haunch.}$$

T. L. per ft. = 800 + 100 = 900#/ft.

$$M = 1.5\, w \cdot L^2 = 1.5 \times 900 \times (20)^2 = 540{,}000''\#$$

$$\frac{I}{c} = \frac{540{,}000}{16{,}000} = 33.7''^3 \qquad \text{Use 12 I 31.8}$$

$$V = \frac{900 \times 20}{2} = 9000\#.$$

$$q = \frac{V}{d_t \cdot t} = \frac{9000}{9.76 \times 0.35} = 2540\#/\square''\ O.K.$$

10,000#/□" allowable.

Limiting span for safe deflection, $L = 2\,d = 2 \times 12 = 24'\text{-}0''$
Actual span = 20'-0" *Deflection O.K.*
Standard beam connections O.K. (Table 25).

9. Flexure.

In designing steel beams to resist bending, the general flexure formula is used, namely,

$$M_r = \frac{s \cdot I}{c}, \quad \text{in which} \qquad (S\text{–}1)$$

M_r = the moment of resistance in inch-lbs.,
s = the maximum allowable fibre stress in #/□",
I = the moment of inertia of the cross-section in $(\text{ins.})^4$,
c = the distance of the extreme fibre from the neutral axis, in inches, and
$\frac{I}{c}$ = the section modulus in $(\text{ins.})^3$.

Structural steel is required to meet a standard test for its physical properties, particularly as to its strength, upon which s is based.

SPECIFICATION CLAUSES *

Tension Tests 5. (*a*) The material shall conform to the following requirements as to tensile properties:

Properties Considered	Structural Steel	Rivet Steel
Tensile strength, lb. per sq. in.	55,000–65,000	46,000–56,000
Yield point, min., lb. per sq. in.	0.5 tens. str.	0.5 tens. str.
Elongation in 8 in., min., %..	$\frac{1{,}400{,}000\dagger}{\text{tens. str.}}$	$\frac{1{,}400{,}000}{\text{tens. str.}}$
Elongation in 2 in., min., %..	22	....

† See sec. 6.

(*b*) The yield point shall be determined by the drop of the beam of the testing machine.

Modifications in Elongation 6. (*a*) For structural steel over ¾ inch in thickness, a deduction of 1 from the percentage of elongation in 8 inches specified in sec. 5 (*a*)

* American Society for Testing Materials, Serial Designation A15–14.

shall be made for each increase of ⅛ inch in thickness above ¾ inch to a minimum of .18 per cent.

(*b*) For structural steel under $\frac{5}{16}$ inch in thickness, a deduction of 2.5 from the percentage of elongation in 8 inches specified in sec. 5 (*a*) shall be made for each decrease of $\frac{1}{16}$ inch in thickness below $\frac{5}{16}$ inch.

Since the ultimate tensile strength given above varies, some working stress, s, had to be established, on the basis of a factor of safety of 4. Accordingly, the maximum **allowable fibre stress** is almost universally taken as **16,000#/□″**.* If the maximum external bending moment in inch-pounds is divided by this stress, the required section modulus is found. Algebraically,

$$\frac{M}{s} = \frac{\text{(ins.) (lbs.)}}{\dfrac{\text{(lbs.)}}{\text{(ins.)}^2}} = \frac{\text{(ins.) (lbs.) (ins.)}^2}{\text{(lbs.)}} = \text{(ins.)}^3 = \frac{I}{c}$$

The section moduli of all the various structural shapes have been calculated. These are established in the customary way (Art. 3) and they are based upon the gross sections. All the values are listed in the structural handbooks, but, for convenience, the values corresponding to the more commonly used shapes are given in Chap. 1.

When the required value of $\frac{I}{c}$ is known, the object of the design is to select a beam which meets this value. The principal feature is to select a beam of the least weight to carry the load. Other factors may control the selection, such as headroom, allowable deflection, and the lack of lateral support; it may also be desirable to keep a given number of beams the same depth. When there are only slight differences in the section moduli required for several beams, one size of beam which will suffice for all may be feasible for many reasons. The availability of certain sizes compared with others should also influence the selection. Ordinarily, the minimum weights of any given depth are the sizes most commonly found in stock (Art. 3).

Illustrative Prob. 9a. Select an I beam to safely resist a maximum external bending moment of 330,000″#. Use s = 16,000#/□″.

$$\frac{M}{s} = \frac{I}{c} \quad \text{or} \quad \frac{330{,}000}{16{,}000} = 20.6''^3.$$

Use a 10 I 25.4.

(Refer to Table 4.)

I beams are by far the most commonly used for simple steel flexural members, as they are well balanced in the sense of having the metal properly distributed. Channels are not as economical in weight because the flanges extend only to one side, and they require more lateral support. They are employed however in special cases such as in framing around openings when a clear face is desired on the opening side, or where it is desired to keep the flange width small so that it may be concealed in partitions, and so on. A single angle or a pair of angles may be used for short spans and light loads, such as for lintels. Tees and zees are occasionally used in stair landing work, and the like.

Illustrative Prob. 9b. Select a channel to carry a load of 400#/ft. on a 12′-0″ span.

Load = 400 $\quad M = 1.5\,w \cdot L^2 = 1.5 \times 460 \times (12)^2$
Bm & Haunch = 60 $\quad M = 99{,}200''$
460#/ft.

$$\frac{M}{s} = \frac{I}{c} \quad \text{or} \quad \frac{99{,}200}{16{,}000} = 6.2''^3.$$

Use 8 ⊏ 11.5.

(Refer to Table 7.)

If the required section modulus is great, no standard beam which is rolled may supply the need. Larger sized Bethlehem beams or Bethlehem girder beams may be used in such cases. For average loads, either standard or Bethlehem beams may be used, depending upon the circumstances (Art. 3). For a given job, one kind of beam (standard or Bethlehem) is generally used throughout. Occasionally plates are added to the top and bottom flanges of beams to increase the section modulus (riveted beam girders, Art. 34). If none of the former types of steel beams furnish the required material, some other form of compound beam Chap. 4), or plate girder (Chap. 5) will have to be used.

Illustrative Prob. 9c. Select a steel I beam to span 40′-0″ and carry a load of 2000#/ft.

Load = 2000 $\quad M = 1.5\,w \cdot L^2 = 1.5 \times 2200 \times (40)^2$
Bm & Haunch = 200 $\quad = 5{,}280{,}000''\#$
2200#/ft.

$$\frac{M}{s} = \frac{I}{c} = \frac{5{,}280{,}000}{16{,}000} = 330''^3.$$

No standard beam is available (Table 4).
Use 30 BI 121.
(Refer to Table 5.)

Prob. 9d. Select an I beam to carry a load of 8000# concentrated at the middle of a 16′-0″ span.

Prob. 9e. What size of I beam is required for a load of 1100#/ft. on a 14′-0″ span?

Prob. 9f. Select a channel to carry two loads of 4000# each, applied at the third-points of a 15′-0″ span.

Prob. 9g. What size of beam is required to carry a load of 2100#/ft. on a 42′-0″ span?

Prob. 9h. Select a 5 × 3½ angle to carry a load of 300#/ft. on a 6′-0″ span. What thickness of 2-3½ × 2½ angles (long legs vertical) may be used? What size of a T-iron may be used? Substitute for the T with a Z-bar.

10. Weakening Effect of Flange Holes.

Holes are often punched or drilled in the flanges of beams, and when they occur at or near the point

* Recently advocates of a fibre stress of 18,000#/□″ have effected changes in some building codes to correspond. As the usual elastic limit of steel is 35,000#/□″, such a stress would reduce the real factor of safety to less than 2. The authors are very anxious to emphasize their objections to such practice where live loads approach actual conditions.

of maximum moment, their effect upon the strength of a beam should be considered. The sectional area of the beam lost by punching the holes lessens the section modulus. A hole in the tension flange reduces the tensile strength because the net area is less and the rivet or bolt does not form a continuity for stress. The strength of the flange in Fig. 12 (*b*) would be less than that of (*a*), for the metal could pull away from the fastenings and tear across the holes, as shown by the heavy lines. The effect of a hole in the compression flange is different. In Fig. 12 (*c*), the more the compression, the more the metal would tend to push on the rivet or bolt If the bolt were a tight fit, the safe compressive

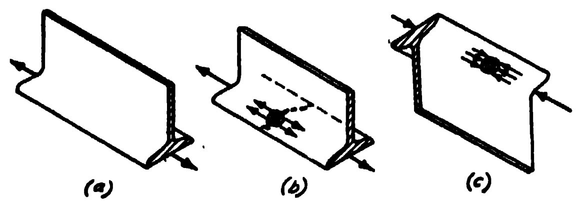


FIG. 12

strength of the flange would be unaffected. It is, however, better practice to consider both the compression and tension flanges on the basis of net area for the following reasons:

(1) The bolt or rivet may not be a tight fit.
(2) The shearing strength of a bolt is less than its compressive strength.
(3) The quality of metal in bolts is often not equal to that in steel beams.

Holes in flanges should be located preferably at a sufficient distance away from the point of maximum moment so that their weakening effect may be neglected. The usual beam is so designed. The actual section modulus supplied is generally somewhat above that required, so that a certain amount of protection is afforded in the usual case. When this cannot be done, the provision for reduced

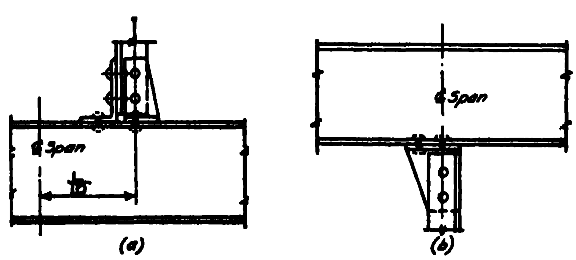


FIG. 13

area must be considered. Figure 13 (*a*) illustrates the limits within which provision should be made for this contingency. Figure 13 (*b*) shows the usual cause for tension flange holes. It may more often rest with the detailer than with the designer to check these cases. Table 13 shows the reduction of strength for various sections under varying conditions of flange punching. Figure 14 gives a diagram by which any condition may be checked.

TABLE 13*

REDUCTION OF STRENGTH IN BEAM SECTIONS DUE TO HOLES IN FLANGES

Beams or Channels	Actual Diameter of Bolt, In.	Actual Diameter of Hole, In.	Percentage of Reduction, One Hole in Flange	Percentage of Reduction, Two Holes in One Flange	Percentage of Reduction, One Hole in Each Flange	Percentage of Reduction, Two Holes in Each Flange
Beams In. Lb						
6 × 12.25	[illegible]	[illegible]	15 9	31.5	18.1	36 1
7 × 15	[illegible]	[illegible]	14 4	28 1	16 1	32 2
8 × 18	[illegible]	[illegible]	14 6	30.0	17 5	35 0
9 × 21	[illegible]	[illegible]	13 9	27 7	16 1	32.3
10 × 25	[illegible]	[illegible]	12 5	25.1	14 7	29.5
12 × 31 5	[illegible]	[illegible]	11 3	22 6	13 4	26.7
15 × 42	[illegible]	[illegible]	9 8	19 6	11.8	23.6
18 × 55	[illegible]	[illegible]	10 0	20 0	12.2	24.4
20 × 65	[illegible]	[illegible]	9 6	19.2	11.7	23 4
24 × 80	[illegible]	[illegible]	8 6	17 2	10.4	20.8
Channels						
6 × 8	[illegible]	[illegible]	23.2	....	28 2	....
7 × 9 75	[illegible]	[illegible]	21.5	....	26 3	....
8 × 11 15	[illegible]	[illegible]	23.5	...	28 6	....
9 × 13.25	[illegible]	[illegible]	20.1	....	24.7	....
10 × 15	[illegible]	[illegible]	20.1	....	25 0	....
12 × 20.5	[illegible]	[illegible]	19 8	....	24 4	....
15 × 33	[illegible]	[illegible]	16 0	....	20 1	....

Illustrative Prob. 10a. An 18 I 55 carries a total uniformly distributed load of 47,100# on a 20'-0" span. This will produce a fibre stress of 16,000#/□" at the center of span. Suppose it is desired to take two holes out of the tension flange at a distance of 4'-8" from the center. Will this be safe?

In Fig. 14 (*a*), follow the curve under the sketch indicating two holes out of one flange of an I beam until it intersects the horizontal line indicating an 18" beam. Project vertically downward from this point to the curve for load uniformly distributed in (*b*). Then project across to the left and note the corresponding percentage of span length. This is 0.23.

$$0.23 \times 20 = 4.6'.$$

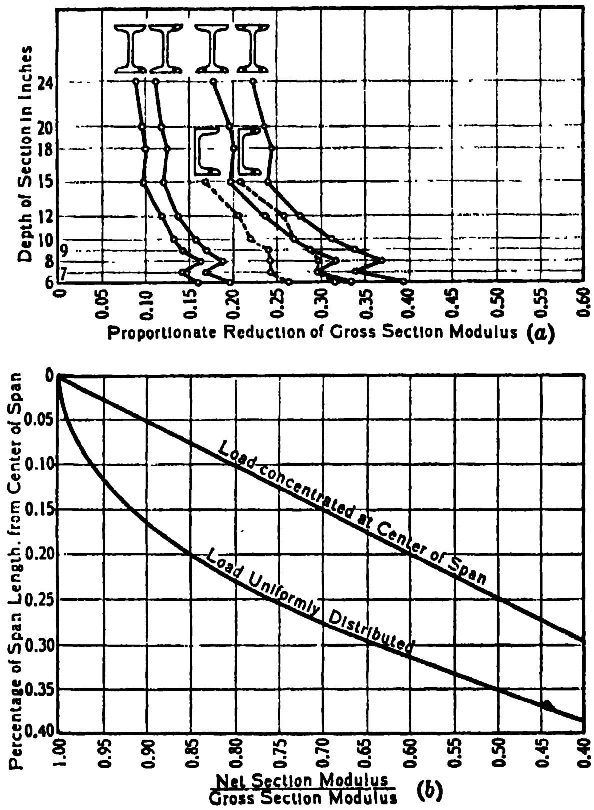
This is less than 4'-8" so that two holes may be taken out as planned.

For any other condition of loading than those indicated in Fig. 14, the values may be approximated without undue error. If the beam in question is one other than the minimum weight of a given depth, the percentage may be obtained by

* Article by R. Fleming of the American Bridge Co., p. 1040, Engineering News Record, March 27, 1920. Only minimum weights of sections are given and the corresponding maximum diameters of bolts allowable. The diameters of the holes deducted are 1/8" greater than the diameters of the bolts in these calculations. The areas for deduction are based on the grips of the bolts.

inverse proportion to the gross section moduli* of this beam and that of the minimum weight beam of the same depth. Thus

$$\frac{\text{Gross } \frac{I}{c} \text{ of Beam in Question}}{\text{Proportion Corresponding to Minimum Weight}} = \frac{\text{Gross } \frac{I}{c} \text{ of Minimum Section}}{\text{Proportion Desired}}.\dagger$$



WEAKENING EFFECT OF FLANGE HOLES

FIG. 14‡

Illustrative Prob. 10b. Find the percentage of the span length for a 15 I 50 with one hole out of one flange. Uniform Load.

I/c (gross) for 15 I 42.9 = 58.9″³
I/c (gross) for 15 I 50 = 64.5″³
% from Fig. 14 (a) for 15 I 42, one hole out of one flange = 0.097

$$\frac{64.5}{.097} = \frac{58.9}{x}$$

$$x = .088.$$

Tracing through the diagram as before,

$$\text{Proportion} = 0.15.$$

* Gross section modulus is defined as that of the whole section. Net section modulus is that of the section with the effect of holes considered.
† This is true within small limits.
‡ Article by H. Kercher of King Bridge Co., p. 799, Engineering News Record, May 12, 1921.

Prob. 10c. A 15 I 42.7 carries a total uniformly distributed load of 36,000# on a 16′-0″ span. It is desired to take two holes out of the tension flange at a distance of 3′-0″ from mid-span. Will this be safe?

Prob. 10d. A 12 I 31.8 has a load of 11,000# concentrated at the middle of a 13′-0″ span. Can two holes be taken out of the top and bottom flanges at a distance of 2′-0″ from the center-line of span?

11. Beams Unsupported Laterally.

If a beam does not have its top flange supported laterally, it will tend to bend in a sidewise direction. Such action will induce stress in addition to that caused by the vertical bending, and therefore the maximum allowable fibre stress in such a case should be reduced. As the top flange is in compression, it acts more or less like a column. The most logical manner to reduce the allowable stress is hence that of employing a column formula. Since there are many column formulas, there are, correspondingly, many formulas for the allowable stresses in unstayed beams. The most commonly used formula for this work is:

$$p = 16{,}000 - 70\frac{l}{r}, \quad \text{in which}$$

p = the maximum allowable compressive stress in #/□″,
l = the unsupported length of the member in inches, and
r = the minimum radius of gyration of the cross-section in inches.

A column will tend to bend in the direction of its least dimension. The great majority of the fibre stress is carried by the flange, so that for simplicity it will be assumed that it is all carried by this part of the beam. Assuming that the flange is a rectangle of any length, x, as in Fig. 15,

$$I = \frac{x \cdot b^3}{12} \quad \text{and} \quad A = x \cdot b.$$

$$r^2 = \frac{I}{A} = \frac{x \cdot b^3}{12} \div x \cdot b = \frac{b^2}{12}, \quad \text{and}$$

$$r = \frac{b}{3.46} \quad \text{(Therefore independent of } x\text{).}$$

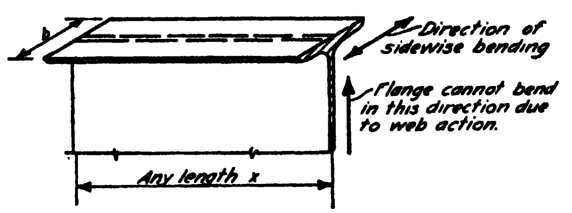


FIG. 15

Substituting in the column formula,

$$p = 16{,}000 - \frac{70\,l}{\frac{b}{3.46}}.$$

Let p in this case be called s_u, the maximum allowable fibre stress when the beam is laterally unsupported. Then

$$s_u = 16{,}000 - 242\frac{l}{b}.$$

It is generally agreed that the coefficient 242 is unnecessarily severe in the majority of cases for the following reasons:

(1) on account of the assumptions previously made,
(2) the maximum stress occurs only at the outer fibres of the flange,
(3) the maximum stress usually occurs at one point along the length only, and
(4) on account of the strengthening action of the web.

Accordingly, this coefficient is reduced to 200. Therefore,

$$s_u = 16{,}000 - 200\frac{l}{b}.^{*} \qquad (S\text{-}2)$$

Illustrative Prob. 11a. What is the maximum allowable stress for a 12 I 31.8 on a 14′-0″ span if it is laterally unsupported?

$l = 14 \times 12 = 168''$ $\qquad b = 5''$ (Table 4)

$\frac{l}{b} = \frac{168}{5} = 33.6$

$s_u = 16{,}000 - 200 \times 33.6 = 9240\#/\square''.$

Another formula used is that developed from

$$p = 19{,}000 - 100\frac{l}{r}.^{\dagger}$$

As before,

$$p = 19{,}000 - \frac{\frac{100\,l}{b}}{3.46}$$

$$s_u = 19{,}000 - 289\frac{l}{b}, \text{ or in round figures}$$

$$s_u = 19{,}000 - 300\frac{l}{b}.$$

When

$$\frac{l}{b} = 10, \quad s_u = 19{,}000 - 300\,(10) = 16{,}000\#/\square''.$$

This value is the same as the usual allowable fibre stress. Consequently no reduction of stress is made when the ratio of span length to flange width is 10 or less. This value is taken for many of the formulas. A limiting value of $l/b = 40$ is often assumed. Beyond such a ratio, no economical design could result.

SPECIFICATION CLAUSE

Limiting Ratios If the ratio of the unbraced length of a beam or girder to its flange width exceeds 10, the maximum allowable fibre stress shall be reduced from the usual by a standard reduction formula. In no case shall the ratio exceed 40.

Still another formula which may be used is developed from

$$p = \frac{S}{1 + \frac{q \cdot l^2}{r^2}} \quad \text{(The Rankine column formula).}$$

$18{,}000\#/\square''$ is used for S because only a portion of the flange is stressed to the maximum. The constant, q, is taken as $\frac{1}{36{,}000}$, that for a fixed end column.

Substituting,

$$p = \frac{18{,}000}{1 + \frac{l^2}{36{,}000\,r^2}}^{\ddagger} \text{ or,}$$

$$s_u = \frac{18{,}000}{1 + \frac{l^2}{36{,}000\frac{b^2}{12}}}$$

$$s_u = \frac{18{,}000}{1 + \frac{l^2}{3000\,b^2}}.$$

If the maximum value of s_u is to be $16{,}000\#/\square''$, then

$$16{,}000 = \frac{18{,}000}{1 + \frac{l^2}{3000\,b^2}}$$

$$16{,}000 + \frac{16{,}000\,l^2}{3000\,b^2} = 18{,}000$$

$$\frac{16{,}000\,l^2}{b^2} = 6{,}000{,}000$$

$$\frac{l^2}{b^2} = \frac{6{,}000{,}000}{16{,}000} = 375$$

$$\frac{l}{b} = 19.4.$$

The Cambria specification therefore requires no reduction of stress when $\frac{l}{b} = 20$ or less. This specification also sets the maximum ratio at 110.

Various building codes and specifications state the limiting ratios for l/b and often the formula upon which the stresses shall be reduced. Table 14 is given as an illustration.

Unless otherwise specified, the first formula given in Table 14 is recommended for determining the allowable stresses in laterally unsupported beams. A point of inconsistency arises when a designer uses one formula to design columns and a formula to calculate allowable stresses for beams laterally unsupported, based upon some other

* If a beam is under a constant moment for a portion of its length, as some loading conditions produce, the formula $s_u = 16{,}000 - 242\frac{l}{b}$ should be used. See Bull Univ. of Ill. #68 and Engineering News Record, May 5, 1916.

† American Bridge Co. column formula.

‡ Rankine column formula used by Cambria Steel Co.

column formula. To be consistent, the same column formula should be basic to all calculations.

TABLE 14

MAXIMUM ALLOWABLE UNIT FIBRE STRESSES FOR BEAMS WITHOUT LATERAL SUPPORT

Ratio $\frac{l}{b}$	A.R.E.A. $16{,}000 - 200\frac{l}{b}$	Fleming $19{,}000 - 350\frac{l}{b}$	A.B. Co. Carnegie $19{,}000 - 300\frac{l}{b}$	Cambria $\frac{18{,}000}{1+\frac{l^2}{3000\,b^2}}$	Bethlehem
5	15,000	16,000	16,000	16,000	16,000
10	14,000	16,000	16,000	16,000	16,000
15	13,000	15,250	14,500	16,000	16,000
20	12,000	14,000	13,000	16,000	16,000
25	11,000	12,750	11,500	14,900	...
30	10,000	11,500	10,000	13,850	14,400
35	9,000	10,250	8,500	12,780	...
40	8,000	9,000	7,000	11,740*	12,800†

The design of beams which are laterally unsupported must be carried along by a "cut and try" method because the width of the beam flange controls the maximum allowable fibre stress, whereas the flange width is not known exactly until the size of the beam required for flexure is established. A trial size may be obtained by selecting a size of beam which will carry the load safely on a span which is laterally supported, and then selecting a size somewhat larger than the first. The latter beam may then be checked to prove that the compressive stress at the extreme fibre is within the allowable and still reasonably near to it, in order to obtain an economical design.

Illustrative Prob. 11b. What size of I beam is required to carry a load of 8000# concentrated at the center of a 14'-0" span if the beam is laterally unsupported?

$$M = \frac{P \cdot L}{4} = \frac{8000 \times 14}{4} = 28{,}000'\# = 336{,}000''\#.$$

$\frac{M}{s} = \frac{336{,}000}{16{,}000} = 21.0''^3$ A 10 I 25.4 would be sufficient if it were laterally supported (Table 4).

Since the beam is unbraced, assume a 12 I 31.8

$$b = 5.0''\quad l = 14 \times 12 = 168''\quad \frac{l}{b} = \frac{168}{5} = 33.6, < 40 \text{ O.K.}$$

$$s_u = 16{,}000 - 200 \times 33.6 = 9240\#/\square''$$

$$\left.\begin{array}{l} \frac{M}{s_u} = \frac{336{,}000}{9240} = 36.4''^3 \text{ required} \\ \frac{I}{c} = 36.0''^3 \text{ actual} \end{array}\right\} \text{O.K. about 1\% over-stressed.}$$

Use 12 I 31.8.

Prob. 11c. What is the ratio of the span length to the flange width for a 15 I 42.9 on a 16'-0" span? Is the ratio within an allowable limit? What is the maximum allowable fibre stress if the beam is laterally unsupported?

Prob. 11d. Design a beam to carry a load of 1000#/ft. on a 15'-0" span if the beam is laterally unsupported.

* Continues $l/b = 110$.

† Continues $l/b = 70$ — Bethlehem beams offer an advantage on account of their wide flanges.

Prob. 11c. Select a channel to carry two loads of 4000# each, concentrated at the third-points of a 12'-0" span, if it is not braced in a sidewise direction.

12. Internal Shear.

Flexure is the usual controlling factor in the design of steel beams, but when beams are of short span with heavy uniform loads, or when there are large concentrations near a support, shear may control the size of beam.‡ The average intensity of vertical shear may be calculated from the formula,

$$v = \frac{V}{A}, \text{ in which}$$

v = the average intensity of vertical shear in #/□",

V = the maximum external vertical shear in #, and

A = the effective area of the cross-section in □".

This shear is assumed to be taken by the area of web only, A_W. This area is the product of the depth of the beam, d, and the thickness of the web, t. Then

$$v = \frac{V}{A_W} = \frac{V}{d \cdot t}. \qquad (S\text{-}3)$$

The intensity of shear at any cross-section varies however, and uniform distribution is only an assumption. It has been shown that the shear is 0 at the outside fibres and maximum at the neutral plane, that it varies as the external shear varies, and that its intensity at any point in a cross-section depends upon the sectional area outside of that point. The intensity of vertical shear and that of horizontal shear at any point are equal. Therefore, to determine the intensity of vertical shear, the equal intensity of horizontal shear may be calculated. The general formula for this calculation is

$$q = \frac{V \cdot Q}{b \cdot I}, \text{ in which}$$

q = the intensity of horizontal shear in #/□",

Q = the statical moment of the cross-section in (ins.)^3, of the area outside of the plane at which q is to be calculated, and

b = the width of the plane of the cross-section in ins., at which q is to be investigated.

The maximum intensity of shear is naturally larger than the average value. In the usual case, the maximum intensity is the only value desired. The width of the beam at the neutral axis is the thickness of the web. The formula then becomes

$$q_{max} = \frac{V \cdot Q}{t \cdot I}. \qquad (S\text{-}4)$$

‡ Mr. R. Fleming gives several instances of this kind in an article in the Engineering News Record, May 27, 1920.

Figure 16 shows the variation of stress in a typical cross-section. It is obvious in (c) that the flanges do not take any appreciable amount of the shearing stress, — hence the reason for assuming that the web takes all of the shear. The exact solution involves the calculation of the statical moment of the cross-section.

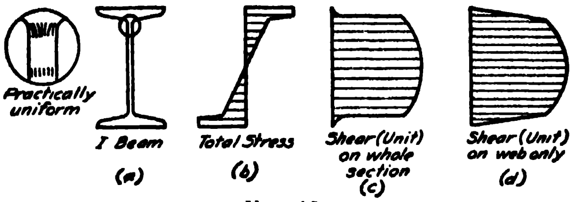


FIG. 16

Illustrative Prob. 12a. What is the average intensity of vertical shear at the support for a 27 I 90 having a maximum end reaction of 120,000#? What is the maximum intensity of horizontal shear?

$$\frac{V}{d \cdot t} \quad \frac{120{,}000}{27 \times 0.524} = 8480\#/\square''.\,^*$$

Calculation of Q. The section should be split up into elementary areas, such as rectangles and triangles, of which the centers of gravity are known. Figure 17 shows the cross-section of the 27 I 90, and the dimensions required. †

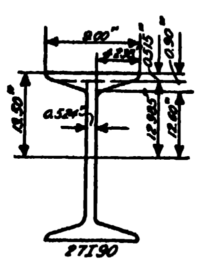


FIG. 17

Q for flange rectangle

$$9 \times 5.15 \times 13.22 = 61.51''^3$$

Q for flange triangles

$$\frac{4.24 \times 0.38}{2} \times 12.86 \times 2 = 20.93$$

Q for web rectangle

$$12.98 \times 0.52 \times 6.49 = 44.22$$

$$\text{Total } Q = 126.66''^3$$

$$I = 2958.3''^4\,†$$

$$q = \frac{V \cdot Q}{t \cdot I} = \frac{120{,}000 \times 126.66}{0.524 \times 2958.3} = 9810\#/\square''.^*$$

Since the maximum allowable shearing stress is 10,000#/□", the beam is satisfactory in this respect.

The above solution is both exacting and laborious if repeated for many cases. Consequently approximate rules of thumb are often used. There are several of these approximations, such as dividing the maximum vertical shear by the area of the web between the flanges, and so on. Of these, probably the most accurate method is to divide the maximum vertical shear, V, by the area resulting from the product of the thickness of the web, t, and the tangent distance on the web, d_t. The tangent distance is that between the points of tangency of the fillets and the web,‡ as in Fig. 18. Algebraically,

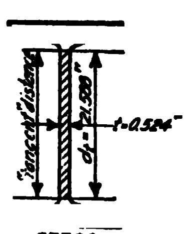


FIG. 18

$$q = \frac{V}{t \cdot d_t}. \qquad (S\text{–}5)$$

The accuracy of such a solution varies, but the results are always on the safe side, the error averaging about 6%.

For practical uses, the allowable shearing stress may be reduced 10% to allow for this variation and thus avoid the frequent references to special detail dimensions of rolled sections.

Illustrative Prob. 12b. What is approximate value of the maximum horizontal shear for the data of Illustrative Prob. 12*a*?

Figure 18 shows the resisting area cross hatched.

$$q = \frac{V}{t \cdot d_t} = \frac{120{,}000}{0.524 \times 21.59}$$

$$q = 10{,}620\#/\square''$$

$$q = 9{,}810 \quad (\text{Prob. } 12a)$$

$$\text{Difference} = 810\#/\square''$$

$$\%\text{ Error} = \frac{810}{9810} = 8.3.$$

In order to avoid approximate solutions, and the corresponding calculations, a table may be developed which can be used to investigate the maximum shear directly, and in addition, corresponds to an exact solution. Thus from

$$q = \frac{V \cdot Q}{t \cdot I},$$

$$V = \frac{q \cdot t \cdot I}{Q}. \qquad (S\text{–}6)$$

The values t, I and Q are constant for any given I beam when referred to the neutral axis (the point where the shear is maximum). The above formula with the maximum allowable value of q substituted in it will therefore express the maximum allowable shear, V, or the end reaction, that a given beam can safely sustain.

The ultimate shearing strength of structural steel averages about 40,000#/□". Based upon the usual factor of safety of 4, the **working stress** quite universally used is **10,000#/□"**. Although the maximum allowable horizontal shear, a function of the vertical shear, is often dependent upon the buckling resistance of the web for its value, the conditions surrounding the usual beam, such as fire-protection and other construction which stiffens the webs of beams, make it safe to assume the fixed value of 10,000#/□" on the web section in place of the lesser value derived by the buckling formulas (Art. 13).

Referring to the data of Illustrative Prob. 12*a* for the 27 I 90, $I = 2958.3''^4$, $Q = 126.66''^3$ and

* The relative values of v and q should be noted.
† See Elements of Sections, Carnegie Pocket Companion.
‡ Tangent distances are given in the dimensions of rolled shapes in structural handbooks.

$t = 0.524''$, the maximum allowable shear on the above basis is

$$V = \frac{10{,}000 \times 0.524 \times 2958.3}{126.66} = 122{,}400\#.$$

Table 15 gives the values of Q and V for various sizes of beams, calculated in the same manner as just illustrated. Use of this table is made by comparing the maximum end reaction obtained in the calculations with the value of V listed which corresponds to the size of beam as selected for flexure. If the reaction is less than the value of V given, the maximum intensity of horizontal shearing stress is within allowable limits. If otherwise, a beam may be selected from the table to meet this requirement.

TABLE 15

MAXIMUM ALLOWABLE SHEARS*

Bethlehem I Beams			Bethlehem Girder Beams		
Size	Q (ins.)^3	V (lbs.)	Size	Q (ins.)^3	V (lbs.)
30–121.0	200.19	141,500	30–200	333.80	208,000
28–106.0	162.90	123,300	181	308.95	182,500
26– 91.0	130.70	109,900	28–165	264.74	161,200
24– 83.0	108.16	107,000	26–151	224.31	145,000
13.5	107.32	99,500	24–121	159.50	121,400
20– 69.0	72.72	90,700	20–113	131.50	97,900
64.5	68.02	80,800	18– 93	92.23	76,500
59.5	64.92	79,400	15–141	122.30	104,300
18– 59.0	59.60	76,600	105	98.93	76,600
54.5	54.22	63,700	74	65.46	58,000
52.0	52.46	54,700	12– 70.5	50.18	48,700
49.0	50.10	51,000	55.5	40.16	39,800
15– 46.0	37.40	55,500	Structural Channels		
41.0	33.73	46,000	15–33.9	25.20	49,900
38.5	33.12	39,000	12–20.7	19.83	39,600
12– 36.5	25.22	33,100	10–15.3	7.91	20,400
32.0	22.42	34,000	9–13.4	6.21	17,500
28.5	20.12	26,800	8–11.5	4.77	15,000
10– 28.5	15.48	33,900	7– 9.8	3.56	12,400
23.5	13.74	24,000	6– 8.2	2.46	10,800
Standard I Beams					
27– 90	126.66	122,400	12–40.8	26.09	47,400
24–100	123.56	142,900	31.8	20.75	36,300
79.9	101.39	103,100	10–25.4	13.98	27,000
20– 81.4	88.71	99,400	9–21.8	10.91	22,600
65.4	68.41	85,500	8–18.4	8.15	18,800
18– 54.7	52.11	72,100	7–15.3	5.94	15,200
15– 42.9	34.29	52,800	6–12.5	4.17	12,000

In special investigations it may be desirable to calculate the intensity of **horizontal shear at points** in a beam which are **not in its neutral plane.** The general shear formula,

$$q = \frac{V \cdot Q}{b \cdot I},$$

is again applicable here, although it is generally used to obtain the maximum shearing stress. The value of Q in this particular case is the statical moment of only that portion which lies outside of the plane where the shear is to be investigated. The value b is the breadth of the section at such a plane.

* Based upon a maximum allowable shearing stress of 10,000#/□".

Illustrative Prob. 12c. Calculate the intensity of horizontal shear at the plane a–a in Fig. 19 if $V = 70{,}000\#$.

Flange rectangle	$7 \times 0.60 \times 11.7$	$= 49.2$
Flange triangles	$\frac{3.25 \times 0.542}{2} \times 2 \times 11.22$	$= 19.7$
Rectangle of part web	$9.15 \times 0.50 \times 6.83$	$= 31.3$
		$Q = 100.2''^3$

$$q = \frac{V \cdot Q}{t \cdot I} = \frac{70{,}000 \times 100.2}{0.50 \times 2087.2} = 6700\#/\square''.$$

Prob. 12d. Check the value of the statical moment, Q, of a 12 I 31.8 about its neutral axis, as shown in Table 15. Check the value of V.

Prob. 12e. If a 15 I 42.9 is subjected to a maximum external vertical shear of 50,000#, what is the average intensity of vertical shear? Is the maximum intensity of horizontal shear safe?

Prob. 12f. Design a beam to carry a load of 1200#/ft. on an 18'-0" span. Check the shear by the approximate method.

Prob. 12g. What is the maximum intensity of horizontal shear at a plane 6" down from the top of a 15 [33.9 which is subjected to a shear of 21,000#?

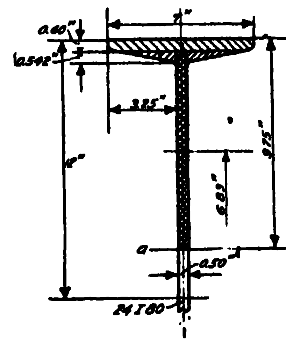


FIG. 19

13. Buckling.

If a large concentrated load or a heavy reaction is applied to a beam, the web may tend to bend sidewise, or buckle (called crippling of the web). In Fig. 20, the two beams shown have the same area of web, and each will carry a certain load as far as the shear is concerned. However, the deeper beam has a greater tendency to cripple. This involves typical column action and therefore a formula for steel columns, expressed in a modified way, may be used to establish

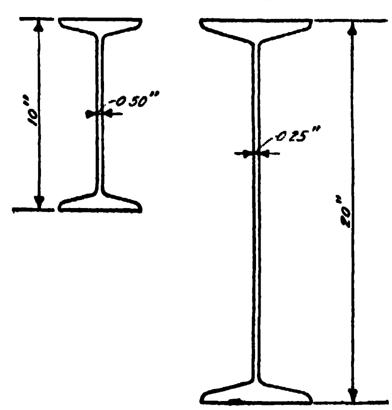


FIG. 20

safe working stresses for such web action. A column formula which is very often used is

$$p = 16{,}000 - 70\frac{l}{r}. \qquad (1)$$

The flanges of an I beam are relatively wide and heavy compared with its web, and therefore they develop the tendency of a fixed end column. In such a member, the points of inflection are assumed to be at the quarter-points of the beam depth, as shown in Fig. 21. The length in which sidewise bending takes place is hence assumed as $\frac{d}{2}$. Substituting $l = \frac{d}{2}$ in (1),

$$p = 16{,}000 - \frac{70\,d}{2\,r}. \qquad (2)$$

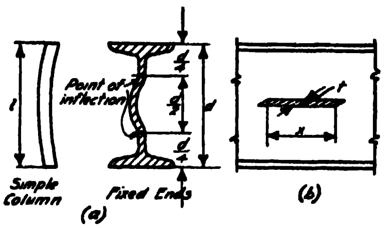


FIG. 21

The resisting section may be assumed to be any length, x. This value will not have any influence on the bending, as a column always tends to bend in the direction of its least dimension, and as shown below, it cancels in the calculations. The value of the radius of gyration, r, is evolved in the following manner:

$$I = \frac{x \cdot t^3}{12}, \quad A = x \cdot t, \quad \text{and}$$

$$r^2 = \frac{I}{A} = \frac{x \cdot t^3}{12} \div x \cdot t = \frac{t^2}{12}, \quad \text{or}$$

$$r = \frac{t}{3.46}.$$

Substituting for r in (2),

$$p = 16{,}000 - \frac{70\,d\,(3.46)}{2\,t}. \qquad (3)$$

Let p in this application be f_b, the maximum allowable buckling stress in pounds per square inch. Then

$$f_b = 16{,}000 - \frac{121.5\,d}{t} \qquad (4)$$

$$\text{or} \quad f_b = 16{,}000 - \frac{120\,d}{t} \text{ (practically).} \qquad (S\text{–}7)$$

Illustrative Prob. 13a. What is the allowable buckling stress for a 12 I 31.8?

$d = 12''$ $t = 0.35''$ (Table 4)

$$f_b = 16{,}000 - \frac{121.5 \times 12}{0.35} = 11{,}830\#/\square''.$$

Since the values of the buckling stress are dependent upon some steel column formula other expressions may be developed for such allowable stresses if other column formulas were used. Thus, using

$$p = 19{,}000 - 100\frac{l}{r},\dagger$$

$$f_b = 19{,}000 - \frac{173.2\,d}{t}, \quad \text{or}$$

$$f_b = 19{,}000 - \frac{170\,d}{t} \text{ (practically).} \qquad (S\text{–}8)$$

Table 16 gives allowable values of f_b for various beams, based upon the two above formulas. To be consistent, the designer should use a buckling stress formula which corresponds to the column formula employed for the general design. Unless otherwise specified, formula (S–7) is recommended.

Buckling should be investigated at the critical points of loading, which are at the end supports of beams that have unrestrained webs, and at points where concentrated loads occur. If the stress is

TABLE 16

ALLOWABLE BUCKLING STRESSES

Bethlehem I Beams			Bethlehem Girder Beams		
Size	(I) $f_b\#/\square''$*	(II) $f_b\#/\square''$*	Size	(I) $f_b\#/\square''$*	(II) $f_b\#/\square''$*
30–121.0	9,340	9,420	30–200	11,300	12,200
28–106.0	9,280	10,300	181	10,800	11,500
26– 91.0	9,280	10,300	28–165	10,900	11,700
24– 83.0	10,480	11,030	26–151	11,000	11,800
73.5	8,660	8,330	24–121	10,700	11,300
20– 69.0	11,440	12,420	20–113	11,700	12,800
64.5	10,720	11,380	18– 93	11,500	12,500
59.5	9,640	9,820	15–141	13,700	15,800
18– 59.0	11,680	12,760	105	13,000	14,670
54.5	10,720	11,380	74	11,910	13,100
52.0	10,240	10,700	12– 70.5	12,930	14,570
49.0	9,250	10,200	55.5	12,210	13,420
15– 46.0	11,910	13,100	Structural Channels		
41.0	10,710	11,350	15– 33.9	11,440	12,510
38.5	9,800	10,050	12– 20.7	10,790	11,570
12– 36.5	11,350	12,300	10– 15.3	10,960	11,780
32.0	11,700	12,750	9– 13.4	11,250	12,220
28.5	10,250	10,700	8– 11.5	8,580	12,700
10– 28.5	12,920	14,550	7– 9.8	11,950	13,230
23.5	11,300	12,080	6– 8.2	12,360	13,810
Standard I Beams					
27– 90	9,830	10,080	12– 40.8	12,870	14,480
24–100	12,140	13,490	31.8	11,890	13,060
79.9	10,250	10,690	10– 25.4	12,120	13,410
20– 81.4	12,000	13,230	9– 21.8	12,280	13,620
65.4	11,200	12,080	8– 18.4	12,440	13,870
18– 54.7	11,310	12,220	7– 15.3	12,640	14,150
15– 42.9	11,110	12,670	6– 12.5	12,900	14,480

* (I) values correspond to $f_b = 16{,}000 - 120\frac{d}{t}$.

(II) values correspond to $f_b = 19{,}000 - 170\frac{d}{t}$.

† American Bridge Co. formula.

proven to be safe at the larger end reaction in such a case, and under the largest concentrated load, the investigation will be complete for any given beam, there being no variation in the web.

At **end supports** a common assumption in design is that the effect of a vertical load spreads at an angle of approximately 60° with the horizontal, as illustrated in Fig. 22. The distance ab is assumed as $\frac{d}{2}$. This will approximate a 60° angle as shown. The average cross-section, A, shown shaded, is taken to be the effective resisting area. This assumption is reasonable because the greatest tendency toward sidewise bending of the web will occur there, as shown in Fig. 21. The allowable resistance to the reaction is the product of the resisting area and the allowable buckling stress, f_b. Hence

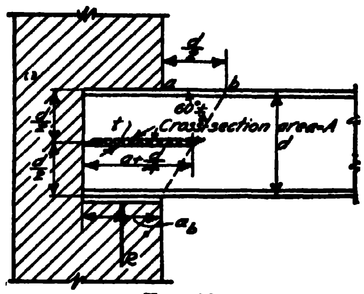


Fig. 22

$R = f_b \cdot A$, in which

$A = \left(a_b + \frac{d}{4}\right) \cdot t$ (the average resisting area).

The safe end reaction, based upon the allowable buckling stress, is therefore

$$R = f_b \cdot t \cdot \left(a_b + \frac{d}{4}\right). \qquad (S\text{–}9)$$

Illustrative Prob. 13b. If a 12 I 31.8 has an 8″ wall bearing (a_b), what is the maximum allowable end reaction as controlled by buckling?

$a_b = 8''$ $d = 12''$ $t = 0.35''$ $f_b = 11{,}830\#/\square''$ (Prob. 13*a*).

$$R = 11{,}830 \times 0.35 \times \left(8 + \frac{12}{4}\right) = 45{,}500\#.$$

The length of the bearing will vary according to the conditions at the support. A **special instance** occurs **when a beam frames into a column,** as illustrated in Fig. 23. The seat angle* is usually a 6″ × 4″, as a standard. The usual clearance allowed between the end of the beam and the face of the column is ½″. The length of the bearing then is 3½′. Using $a_b = 3\frac{1}{2}''$, and substituting in formula (*S*–9),

$$R = f_b \cdot t \cdot \left(3.5 + \frac{d}{4}\right)$$

(*S*–10) (Special for Column Brackets.)

* When a beam has standard connection angles and a seat angle is used for erection purposes only, the buckling need not be investigated because the beam will eventually be clear of the seat angle.

Illustrative Prob. 13c. What is the maximum allowable end reaction for a 12 I 31.8 resting upon a standard beam seat?

$f_b = 11{,}830\#/\square''$ (Prob. 13*a*)

$R = 11{,}830 \times 0.35\left(3.5 + \frac{12}{4}\right)$

$R = 26{,}900\#.$

The **safe concentrated interior load,** based upon the allowable buckling stress, may be derived in a manner similar to that for the safe end reaction. In Fig. 24, the average area, shown shaded, is $t \cdot \left(a_c + \frac{d}{2}\right)$. Then

$$P = f_b \cdot t \cdot \left(a_c + \frac{d}{2}\right) \qquad (S\text{–}11).$$

The formulas for safe end reactions and safe interior loads are sometimes called purely empirical, but the reasoning is rational, and they have a basis in true mechanics. Experimental tests verify these formulas and show them to be reasonably accurate.

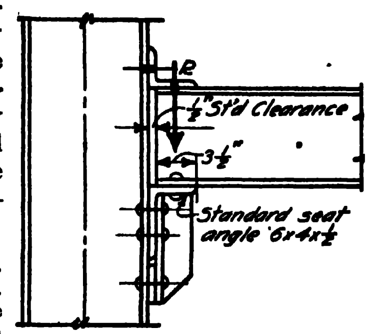


Fig. 23

Illustrative Prob. 13d. If P in Fig. 24 is 50,000# and the length of the column base-plate is 12″, is the buckling safe for a 12 I 31.8?

$a_c = 12''$ $d = 12''$ $t = 0.35''$

$50{,}000 = f_b\,(0.35)\left(12 + \frac{12}{4}\right)$

$f_b = 9550\#/\square''$ actual; (From Prob. 13*a*) $f_b = 11{,}830\#/\square''$ allowable — O.K.

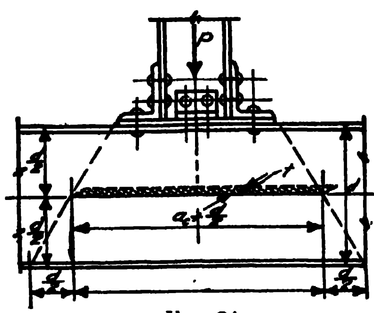


Fig. 24

If the buckling stress as calculated is excessive, a beam with a stiffer web must be used, or the web must be reinforced. Stiffeners may be placed under the loads but their use is not considered economical for rolled beams.

When beams bear on walls and the usual lengths of bearing are employed, namely

for 6″ beams . . . . 6″ or 8″,
7, 8, 9 and 10″ beams . . 8″ or 10″,
12 and 15″ beams . . . . . . 10″, 12″ and 16″,
18″ beams . . . . 12″ or 16″,
beams > 18″ deep. . . . . . 16″,

the safe end reaction, as controlled by buckling, is practically always greater than that controlled by a safe shearing resistance. To illustrate for a 12 I 31.8, the following calculations are given:

$$f_b = 16{,}000 - 120\frac{d}{t}$$

$$= 16{,}000 - \frac{120 \times 12}{0.35} = 11{,}900\#/\square''.$$

$$R = f_b \cdot t \cdot \left(a_b + \frac{d}{4}\right).$$

Even for an 8″ bearing,

$$R = 11{,}900 \times 0.35\left(8 + \frac{12}{4}\right) = 45{,}700\#.$$

From Table 15, the safe end reaction based upon a safe shearing resistance = 36,300#, which is a lesser value.

The same relation may be shown to be true for a number of other cases. If the allowable buckling stress is based upon $f_b = 19{,}000 - 170\frac{d}{t}$, the difference is even more on the safe side. Consequently, unless beams have a shorter length of wall bearing than is usual, the end reaction will not cause excessive buckling stresses in the web if the shear is safe. Since the shearing stresses are generally safe, the **investigation for buckling in such cases is commonly omitted.** This is also more or less true for interior concentrated loads when they are distributed over a reasonable length of beam. In any particular instance of either case, the general procedure previously discussed should be followed out. It should be well understood that the above discussion does not apply when beams are supported on column brackets. In such cases, the maximum allowable reaction is practically always controlled by the safe buckling resistance of the beam web. For this reason, Table 17 is given for the more common sizes of beams.

Another instance of buckling is found in the tendency of the web to cripple at some **point between the supports and concentrated loads.** The usual place is where the shear is a maximum, that is, close to a support. Stress of this nature will usually be within allowable limits, especially if the buckling stress is safe at the support. Some designers prefer, however, to investigate this action for heavily loaded beams, particularly if the ratio of the span to the depth is small.

TABLE 17

SAFE END REACTIONS FOR BEAMS ON COLUMN BRACKETS (BASED ON BUCKLING)

Type	Beam Size	Safe Reaction $f_b = 16{,}000 - 120\frac{d}{t}$	Safe Reaction $f_b = 19{,}000 - 170\frac{d}{t}$	Type	Beam Size	Safe Reaction $f_b = 16{,}000 - 120\frac{d}{t}$	Safe Reaction $f_b = 19{,}000 - 170\frac{d}{t}$
Standard Beams	24–100.0	85,900	96,620	Channels	15–33.9	33,200	36,270
	79.9	48,700	50,780		12–20.7	19,660	21,060
	20– 81.4	61,200	67,460		10–15.3	15,800	16,970
	65.4	47,600	51,320		9–13.4	14,900	16,170
	18– 75.6	58,870	60,210		8–11.5	13,960	15,370
	54.7	41,620	44,080		7– 9.8	13,200	14,580
	15– 60.8	55,400	62,240		6– 8.2	12,360	13,810
	42.9	33,020	37,660				
	12– 40.8	38,420	43,300				
	31.8	26,900	29,710				
	10– 25.4	21,700	24,940				
	9– 21.8	20,500	22,710				
	8– 18.4	18,500	20,600				
	7– 15.3	16,600	18,580				
	6– 12.5	14,820	16,650				
Bethlehem Girder Beams	30–200	94,730	100,230	Bethlehem I Beams	30–121	55,800	56,000
	181	81,750	87,000		28–106	46,500	50,800
	28–165	75,500	85,200		26– 91.0	42,600	42,800
	26–151	69,600	74,800		24– 73.5	32,200	32,000
	24–121	54,900	58,000		20– 73.0	38,250	40,270
	20–113	55,700	61,000		59.5	30,730	31,300
	18– 93	44,200	48,000		18– 49.0	23,760	26,110
	15–141	79,400	91,600		15– 38.5	22,010	27,460
	105	56,550	66,710		12– 32.0	25,500	28,000
	74	38,090	41,790		28.5	16,660	17,400
	12– 70.5	39,500	44,500		10– 28.5	30,200	34,000
	55.5	30,500	33,250		23.5	16,800	18,100

The safe end reactions for beams, based upon buckling **at wall bearings,** are usually in excess of the allowable web resistances. Consequently the buckling for such cases is safe if the shear is safe. (Table 15.)

The same is similarly true for interior concentrated loads.

The shearing stresses acting in the web of a beam may be resolved into component stresses of tension and compression of equal intensity and acting at right angles to each other. The intensity is equal to that of the vertical shear, and near the supports the inclination is practically 45° with the neutral plane and in the plane of the web. The compressive stresses tend to buckle the web but it is not entirely free to buckle because the tensile forces acting upon it have the effect of stiffening it. The web may be considered as a series of columns of a length equal to the diagonal distance on a 45° line between the flanges, as in Fig. 25. Let AB be such a strip 1″ wide. In this case the flanges are again assumed as providing stiffness to

the small columns of steel in the web, and the depth of the beam is used as a basis for calculating the length of the imaginary columns in the web. This length is therefore $d\sqrt{2}$. Assuming fixed end conditions as before, the effective length, l, in the formula $16{,}000 - 70\frac{l}{r}$ is $\frac{d\sqrt{2}}{2} = 0.707\,d$. Thus, when $r = \frac{t}{3.46}$ as before,

$$p = 16{,}000 - 70\frac{0.707\,d}{\frac{t}{3.46}}, \quad \text{or}$$

$$p = 16{,}000 - 172\frac{d}{t}.$$

Using f_b for p in this case,

$$f_b = 16{,}000 - 170\frac{d}{t}\text{ (practically).} \qquad (S\text{–}12)$$

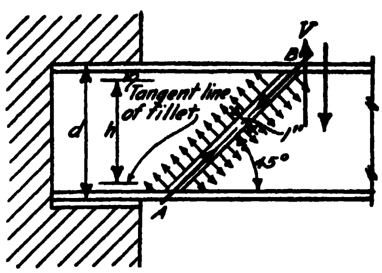


FIG. 25

The allowable buckling stress may be calculated for any given beam by this formula. The actual intensity may be taken as that of the average vertical shear without great error, as it is equal to the average intensity of the compressive stress. The maximum web shear for the beam may be found by assuming f_b to act over an area equal to $d \cdot t$.

Illustrative Prob. 13e. A 15 I 42.8 on a 10′-0″ span has a total uniform load of 62,000#. Is the web safe against buckling?

$$\frac{62{,}000}{2} = 31{,}000\# \quad v = \frac{V}{d \cdot t}\text{ (Art. 12).}$$

$$v = \frac{31{,}000}{15 \times 0.41} = 5040\#/\square''\text{ actual.}$$

$$f_b = 16{,}000 - \frac{170 \times 15}{0.41} = 9770\#/\square''\text{ allowable}^* > 5040.$$

Therefore the beam is amply safe.

Other formulas may be employed. The Cambria Steel Company uses

$$p = \frac{S}{1 + q\frac{l^2}{r^2}}\text{ (Rankine Formula).}$$

With $S = 12{,}000\#/\square''$, and $q = \frac{1}{36{,}000}$ for a fixed end column, $l^2 = 2\,h^2$, and $r^2 = t^2/12$, this becomes (h = clear distance between tangent lines of fillets and is the basis of these formulas)

$$p = \frac{12{,}000}{1 + \frac{2\,h^2}{36{,}000\frac{t^2}{12}}} = f_b.$$

$$f_b = \frac{12{,}000}{1 + \frac{h^2}{1500\,t^2}}.$$

The Bethlehem Steel Company uses the same formula but assumes the effective length of the column as h.

$$f_b = \frac{12{,}000}{1 + \frac{h^2}{3000\,t^2}}.$$

On the basis of $p = 19{,}000 - 100\frac{l}{r}$ (The American Bridge Co. column formula),

$$f_b = 19{,}000 - 242\frac{h}{t}.$$

Prob. 13f. Check the maximum allowable buckling stress for a 15 I 42.9 as given in Table 16.

Prob. 13g. If an 18 I 54.7 has a wall bearing of 10″ and an end reaction of 49,000# is the buckling safe at the support?

Prob. 13h. A 15 I 42.9 has an interior concentrated load of 31,000# applied over a distance of 6″, is the beam safe in buckling?

Prob. 13i. A 12 I 40.8 frames into a 12″ B H column with a standard seat angle detail. The end reaction is 40,000#. Is the buckling safe?

Prob. 13j. A 12 I 31.8 header beam has a load of 60,000# concentrated at the center of a 6′-0″ span. Is the buckling safe between the supports and the load? How does it compare with a maximum allowable shear of 10,000#/□″?

14. Deflection.

The general formulas for deflection derived in mechanics apply to steel beams in the usual way. The values should not exceed $\frac{1}{360}$ of the span in inches. For symmetrical sections carrying uniform load the formula for safe deflection is

$$D = \frac{30\,s \cdot L^2}{E \cdot d}, \text{ in which}$$

D = the deflection in inches,
s = the allowable fibre stress in #/□″,
L = the span of the beam in feet,
E = the modulus of elasticity of the steel, in #/□″, and
d = the depth of the beam in inches.

Rules of thumb may be evolved which are useful as guides in limiting the deflection to safe values. The safe limit for deflection is

$$L = \frac{E \cdot d}{900\,s}.$$

For structural steel, $E = 29{,}000{,}000\#/\square''$ and $s = 16{,}000\#/\square''$. Substituting,

$$L = \frac{29{,}000{,}000\,d}{900 \times 16{,}000} = 2.01\,d.$$

$$L = 2\,d\text{ (practically).}\dagger \qquad (S\text{–}13)$$

* A beam may be safe in vertical shear based on 10,000#/□″ but not always in buckling. See Art. 12.

† It should be remembered that the beam must be fully stressed for this rule to apply.

Stated as a rule,

> **The limiting span in feet for safe deflection, for a simply supported beam uniformly loaded, is equal to twice the depth of the beam in inches.**

Other limiting spans can be solved for in a similar manner. The following tabulation gives some of these values worked out on this basis.

TABLE 18

LIMITING SPANS FOR SAFE DEFLECTION

Kind of Beam	Manner of Loading	Symmetrical Sections	Unsymmetrical Sections
Simply supported	Uniform	$2\,d$	$4\,c$
do	Conc. @ C.L.	$\frac{5\,d}{2}$	$5\,c$
Cantilever	Uniform	$\frac{5\,d}{6}$	$\frac{5\,c}{3}$
do	Conc. @ free end	$\frac{5\,d}{8}$	$\frac{5\,c}{4}$

d = depth of beam in inches, L = limiting span in ft., c = distance from neutral axis to extreme fibre.

For other conditions of loading the deflection may be calculated from the general formulas which are given in the standard books on mechanics.

Illustrative Prob. 14a. Design a beam to carry a total load of 2000#/ft. on a 20'-0" span and show that its deflection is safe.

$$M = 1.5\,wL^2 = 1.5 \times 2000\,(20)^2 = 1{,}200{,}000''\#.$$

$$M = \frac{s \cdot I}{c} = 1{,}200{,}000 = 16{,}000\,\frac{I}{c}$$

$$\frac{I}{c} = 75.0''^3 \qquad 18\text{ I }55,\ \frac{I}{c} = 88.4''^3$$

$$\left\{\begin{array}{l} 15\text{ I }60,\ \frac{I}{c} = 81.2''^3 \\ \text{Use on account of headroom.} \end{array}\right.$$

$$D = \frac{5\,W \cdot l^3}{384\,E \cdot I}$$

$$\left.\begin{array}{r} D = \dfrac{5\,(2000 \times 20) \times (20)^3 \times (12)^3}{384 \times 29{,}000{,}000 \times 600} = 0.41''\text{ actual} \\ D\text{ (allowable)} = \dfrac{20 \times 12}{360} = 0.67'' \end{array}\right\}\text{O.K.}$$

Deflections due to shearing stresses will not be considered here because of their relatively small values. Table 19 shows an interesting comparison of deflections due to flexure and shear.

Prob. 14b. Calculate the maximum deflection for an 18 I 54.9 carrying a load of 1000#/ft. on a 30'-0" span. Is it safe? How does the rule of thumb compare with your results?

Prob. 14c. A 12 I 31.8 has a load of 16,000# concentrated at the middle of its 12'-0" span. Calculate the maximum deflection. Is it safe? Compare with Table 18. What is your conclusion?

Prob. 14d. Design a beam to carry the loads shown in Fig. 26 and calculate the deflection by the approximate method. Is it safe? (Hint: find equivalent uniform load.)

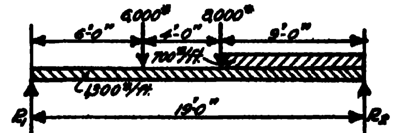


FIG. 26

TABLE 19*

DEFLECTION OF BEAMS: FLEXURE AND SHEAR

I Beams	Span in Ft.	Load in Lbs.	Distribution	Δf in In.	Δs in In.	$\Delta f + \Delta s$ in In.
In. Lb.						
8 × 18	5	30,300	Uniform	0.050	0.011	0.061
8 × 18	10	15,150	Uniform	0.200	0.011	0.211
8 × 18	15	10,100	Uniform	0.449	0.011	0.460
8 × 18	5	15,150	Middle	0.040	0.011	0.051
8 × 18	10	7,600	Middle	0.160	0.011	0.171
8 × 18	15	5,050	Middle	0.359	0.011	0.370
12 × 31.5	5	76,700	Uniform	0.033	0.014	0.047
12 × 31.5	10	38,350	Uniform	0.133	0.014	0.147
12 × 31.5	20	19,175	Uniform	0.534	0.014	0.548
12 × 31.5	5	38,300	Middle	0.027	0.014	0.041
12 × 31.5	10	19,150	Middle	0.107	0.014	0.121
12 × 31.5	20	9,575	Middle	0.427	0.014	0.441
20 × 65	10	124,700	Uniform	0.080	0.019	0.099
20 × 65	20	62,350	Uniform	0.321	0.019	0.340
20 × 65	40	31,175	Uniform	1.281	0.019	1.300
20 × 65	10	62,400	Middle	0.064	0.019	0.083
20 × 65	20	31,200	Middle	0.256	0.019	0.275
20 × 65	40	15,600	Middle	1.024	0.019	1.043

Notation: Δf = deflection due to flexure.
Δs = deflection due to shear.
$\Delta f + \Delta s$ = total deflection.

15. Bearing at Supports.

Another feature in the design of steel beams which must be investigated is that of providing sufficient bearing area at the supports. A beam is supported either by a wall, or framed into another beam or into a column.

At the ends of beams which rest on walls, a bearing plate is provided in the majority of cases. For light end reactions a plate may not be required theoretically, but for practical reasons a plate should be used. This will facilitate the proper seating of the beam at its correct elevation. The plate insures a level surface for the beam to rest upon and prevents the load from being irregularly concentrated upon the toes of the flanges. Cast-iron plates for steel beams are impractical because of the relatively large reactions and the low allowable flexural stress, but they are still used occasionally. In cases where excessive reactions occur, and a single plate is insufficient, a series of small I beams may be used in the form of a grillage, or a rolled steel slab may be employed.†

* From an article by R. Fleming, Engineering News Record, May 27, 1920.
† See Index.

The action of a bearing plate for a steel beam is illustrated in Fig. 27. The bending may be assumed as acting about the center line of the beam, as in (*a*), or about the toe of the flange, as in (*b*). A discussion of these methods and the formulas resulting may be found in many text books on mechanics. These formulas are as follows:

$$M_e = \frac{p\,(B^2 - b^2)}{8} \text{ (About the C.L. of beam), and}$$

$$M_e = \frac{p\,(B - b)^2}{8} \text{ (As a cantilever),} \qquad (S\text{–}14)$$

both for a strip 1″ wide. The dimensions B and b are shown in Fig. 27 and are in inches. The

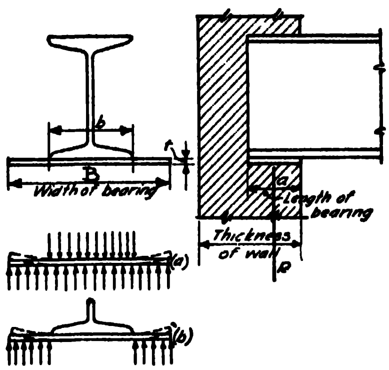


FIG. 27

value p is the actual pressure on the plate in lbs. per sq. in. which is not always the allowable. The thickness of plate may be found from

$$t = \sqrt{\frac{6\,M_e}{s}}. \qquad (S\text{–}15)$$

As beams are ordinarily embedded in the masonry, the plate is not in simple flexure, and s may be taken at 20,000#/□″.

Illustrative. Prob. 15a. Design a bearing plate for a 15 I 42.9 with an end reaction of 40,000#, bearing on a 16″ brick wall set in 1 : 3 P.C. mortar.

The allowable bearing = 250#/□″.

$\frac{40{,}000}{250}$ = 160□″ required. Length of bearing = 16″ − 4″ = 12″ maximum.

$\frac{160}{12}$ = 13.3. B = 14. Try 12 × 14 Plate.

$\frac{40{,}000}{12 \times 14}$ = 238#/□″ = p, the actual pressure.

b for a 15 I 42.9 = 5.5″

$$M_e = \frac{238\,(14 - 5.5)^2}{8} = 2150''\#$$

$$t = \sqrt{\frac{6 \times 2150}{20{,}000}} = 0.81''$$

Use 12 × $\frac{7}{8}$ Pl. × 1′-2″.

If the flanges are relatively wide and thin, such as those of the **Bethlehem beams,** the **critical point** of flexure may not be at the toe of the flange but **at the root of the fillet,** as at plane x–x in Fig. 28. The combined resistance of the plate and flange should be checked in this case. The plate is usually loose so that there is no connection between it and the flange.* Therefore the resistance is the sum of those for each part considered separately, and not based upon the total thickness as if the two parts were riveted. The steel in the Bethlehem beam flanges is perhaps a little stronger than that of standard beams because the former are rolled in a Universal mill (Art. 3), but no allowance is made for such a possible increase.

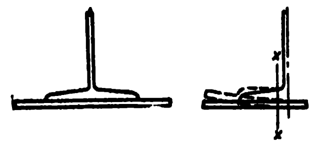


FIG. 28

Illustrative Prob. 15b. Check the thickness of plate in Illustrative Prob. 15*a*, basing the calculation of the moment about the root of the fillet.

Projection beyond plane x–x, Fig. 29 (*a*) is

$$7.00 - \frac{0.41}{2} - 0.51 = 6.29''$$

$$M_e = \frac{238 \times (6.29)^2}{2} = 4720''\#$$

$$M_r \text{ of flange} = \frac{s\,(1)\,t^2}{6} = \frac{20{,}000 \times 1 \times (0.83)^2}{6} = 2300''\#$$

4720 − 2300 = 2420″# to be supplied by the plate.

$$M_r \text{ of } \tfrac{7}{8}'' \text{ plate} = \frac{20{,}000 \times 1 \times (0.875)^2}{6} = 2560''\#$$

12 × $\frac{7}{8}$ Pl. × 1′-2″ O.K., as before.

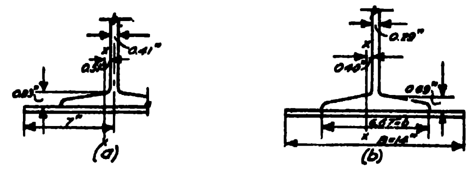


FIG. 29

For standard beams, the thickness of the plate, as determined by its projection beyond the toe of the flange, is usually satisfactory.

Illustrative Prob. 15c. Find the required thickness of the plate for the data of Illustrative Prob. 15*a*, if the beam in question is a 15 BI 38.

From Prob. 15*a*, the size of the plate is 12″ × 14″, and p = 238#/□″.

$$M_e \text{ @ toe of flange} = \frac{238\,(14 - 6.67)^2}{8} = 1600''\#$$

$$t = \sqrt{\frac{6 \times 1600}{20{,}000}} = 0.69'' \qquad \text{Try a } 12 \times \tfrac{3}{4} \text{ Pl.} \times 1'\text{-}2''.$$

It will be noted in comparison with Prob. 15*a* that a thinner plate could be used as far as this calculation is concerned,

* For the design of a bearing plate which is riveted to the bottom flange (sole plate), see Art. 72.

because the flange is wider and consequently the projection less. From the root of fillet (x–x) the projection in Fig. 29 (b) is

$$7.00 - \frac{0.29}{2} - 0.40 = 6.46''$$

$$M_e = \frac{238\,(6.46)^2}{2} = 4970''\#.$$

$$M_r \text{ of flange} = \frac{20{,}000 \times (0.69)^2}{6} = 1590''\#$$

$$M_r \text{ to be supplied} = \overline{3380''\#}$$

$$t = \sqrt{\frac{6 \times 3380}{20{,}000}} = 1.01''$$

Use 12 × 1″ Pl. × 1′-2″.

It is therefore necessary that a thicker plate be used than in Prob. 15a, because the thin flange controls. This is the usual case for Bethlehem beams and the second part of the calculation above may be used at once to find the required thickness. Table 20 gives moments of resistance of various plates.

TABLE 20

MOMENTS OF RESISTANCE OF PLATES PER INCH OF WIDTH

Thickness	s = 16,000#/□″	s = 20,000#/□″
¼	156″#	208″#
⅜	344	464
½	625	834
⅝	975	1300
¾	1400	1870
⅞	1910	2550
1	2500	3330

As the thickness of plates is not controlled by any practical consideration of riveting, they might be made any thickness within reason. However, the thickness used is seldom greater than 1″ for average conditions, as thicker plates are not as common in stock. Stock plates are obtainable in thicknesses varying by ⅛ths of an inch, and $\frac{1}{16}$th variations are not as common. The minimum thickness in any case should not be less than ⅜″ because of the corrosion of thinner plates. The thickness selected should in each instance be the nearest ⅛″ above the value theoretically required. Some common sizes used are:

6 × ⅜ Pl. × 0′-6″, 8 × ½ Pl. × 0′-8″,
8 × ⅝ Pl. × 0′-10″,
8 × ¾ Pl. × 1′-0″, 8 × 1 Pl. × 1′-2″ or 1′-4″,
and so on.

Tables of projection coefficients to design wall plates are given in various structural steel handbooks,* but weight often may be saved by more careful design, as these tables are based on the allowable rather than the actual bearing pressures. These tables are valuable when speed in calculations is imperative.

Where the ends of several beams are to be provided for, the designer should aim to keep the number of sizes of plates a minimum, and use one size of plate for several beams under similar conditions, of course designing for the worst case. The plates may be billed in one corner of the erection diagram in a small schedule for the convenience of the erector. The plates are sometimes given a mark of WP with a number and conventionally represented in their location with their mark. The plate should be called for by its length of bearing by its thickness by its width of bearing, such as:

6 WP 1 — 6 × ⅜ Pl. × 0′-6″,
8 WP 2 — 10 × ⅝ Pl. × 1′-0″, and so on.

It is common practice in some cases to use a standard sized plate for a given beam.* This is done to save time and to avoid the design of details. Standard bearing plates are not always practical to use, because a relatively large beam often has to be carried on an 8″ bearing on account of the architectural features of the wall. Furthermore, the length of the bearing should be in proportion to the size of the beam if possible.

When a beam does not terminate on a wall, details similar to those shown in Fig. 30 may be used. The areas in bearing in (a) and (c) are not usually

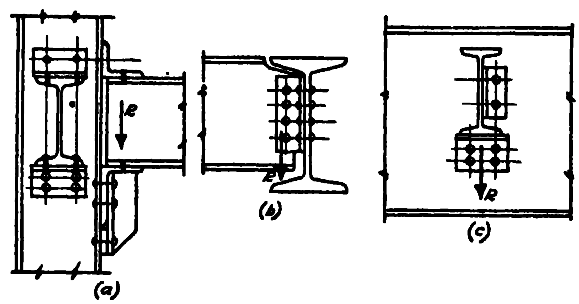


FIG. 30

controlling factors, because the allowable bearing value for steel resting upon steel is relatively very high (about 20,000#/□″), and the area required upon this basis would be small. The safe strength of the rivets will more often limit the allowable end reaction. The detail shown in (b) is not controlled by a bearing support. The design of these types of provisions for end reactions is discussed with that of steel columns and beam connections (see Index).

* See Carnegie "Pocket Companion" — "Bearing Plates."

Prob. 15d. Design a bearing plate for a 24 I 79.9 carrying a load of 85,000# on a 22'-0" span. 16" brick wall, 1 : 3 P.C. mortar.

Prob. 15e. Design a bearing plate for a 30 BI 121 for a load of 51,000# concentrated at the middle of a 36'-0" span. 12" wall bearing, $p = 175\#/\square''$.

Prob. 15f. Design the bearing plates required in Fig. 26 if the beam is supported on 12", 1:2:4 concrete walls. Allowable bearing 450 #/□".

16. Anchorage.

All beams bearing on walls should be securely anchored to the walls to prevent lateral displacement at their ends. The most common type of anchor is shown in Fig. 31 (*a*), while (*b*) shows a type employed in some localities. When beams frame into other beams or into columns, as shown in Fig. 30, sufficient anchorage is provided by the connections. The clip angle anchor, as in Fig. 31 (*b*), affords some protection against web buckling and this feature of beam failure is often disregarded when this anchor is used.

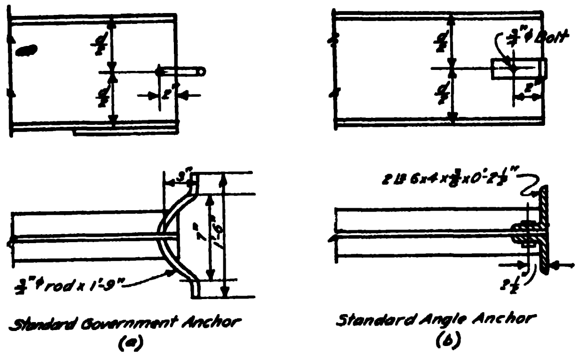


FIG. 31

17. Tie Beams and Strut Beams.

A tie beam is one which is subjected to transverse loads and to direct tension as well, such as a bottom chord member of a truss in certain instances. A strut beam is one which is subjected to transverse loads and to a direct compression as well, such as a top chord member of a truss in certain cases, or a column subjected to lateral forces. A very common use of tie beams is to afford columns bracing at floor lines where a rolled section is not theoretically necessary. Together with the principles of steel beam design just discussed, the design of such beams may be accomplished by employing the methods illustrated in Art. 207.

CHAPTER 3

BEAM CONNECTIONS

18. General.

The rolling mills which furnish the "raw material," that is, the structural shapes, do not generally do the work of cutting the pieces to the desired length, punching the required holes, connecting on the framing angles or plates, cutting away projecting flanges, and so on. This work is done in the plants of separate companies, which are called fabricating shops (see Plate 6). These concerns develop their own shop drawings according to their standard details. Consequently, it is not usually within the province of the structural engineer to supply the actual details which are to be used. However, it is important that the engineer should have a working knowledge of good structural steel details, as a lack of such knowledge adds to the cost of fabrication and erection. Furthermore, a structural engineer is often called upon to approve the structural details as submitted by the fabricating concern, and in such instances, a clear understanding of the methods used will aid in determining the approval of plans and in detecting any errors in them, or other inconsistencies. In many instances, indications of framing made by some structural engineers have been blindly followed by the fabricators, and, as a result, beam flanges have had to be burned off for a given distance, new holes drilled, seat angles cut in order to keep them within the fireproofing, and so on. Thus, it should be evident that while the details seem like a "trivial" matter to some, they are often the most important economic factor in the work. Figure 32 shows some typical beams ready for shipment (see also Pl. 6).

The various parts of structural steel work are held together by rivets, bolts or pins.* Rivets are the most common means of fastening steel, and they are used for beam and girder connections, compounding beams, plate girders, as well as for built-up columns, column connections, trusses, and so on.

Fig. 32

Bolts may be used in certain instances for the same purpose. The following statements will serve to clarify the uses of each:

(1) Bolted connections may be used entirely for one- or two-story buildings which are not of great height and in which the floor construction carries no machinery or moving loads.

(2) They may be used in buildings erected for temporary use.

(3) They may be used in subordinate framing, such as for purlins, girts, stairs and pent houses.

(4) They may be used in larger buildings for connections of beams to beams and for connections of beams to girders, when the floors are not carrying machinery.

(5) They may be used when one member

* Electric welding now offers possibilities.

rests on top of another or in any beam connection which is covered by fire-protecting materials.

(6) They shall not be used in buildings subjected to any considerable amount of wind load.

(7) Rivets must be used in all connections within 3′-0″ of columns, in all connections of beams or girders to columns, in all column and truss details, in bracing connections, and in beam connections for tank or sheave beams or other beams subjected to vibration.

(8) Rivets must never be used where the rivet itself is submitted to direct tension.

Bolts generally do not supply the strength that rivets do, so that for this reason, as well as for those given above, they are used principally for light work and small jobs. The use of pins is confined principally to joints of large trusses and this is discussed in connection with such joints (see Index).

Prob. 18a. What kind of field connections may be used in a two story garage if it is protected from wind pressure by adjacent buildings?

19. The Theory of Riveting.

There is no definite theory, in a fixed sense, for designing riveted joints, and certain assumptions, based upon various tests, are made. These do not all have definite relations to each other. The difference in designing riveted joints for boiler and standpipe work and those for structural work is quite marked. In the former, the joints are designed to obtain the maximum efficiency, that is the same factor of safety is maintained in the net section of the plate (with the holes deducted) and in the strength of the rivets themselves. In other words the theoretical diameter of the rivets is calculated upon this basis and the nearest commercial size of rivet is used. In structural steel work, one or two common sizes of rivets are used throughout, and a sufficient number of them is used to develop the stress in the member, and the rivets are spaced so that the net section is not reduced below an allowable value.

Illustrative Prob. 19a. Compare the two methods of design (boiler plate and structural) for the joints shown in Fig. 33.

In (*a*), the boiler plate joint, the efficiency is equal to the ratio of the strength at the weakest point of the plate to the strength of an equal length of a solid plate. The failure by shearing or crushing of the rivets is equated to that governed by the tearing of the plate, so that there is never an excess of strength in the tearing resistance.

Let f_s = the shearing strength of the rivets, say 12,000#/□″
f_t = the tearing strength of the plate, say 16,000#/□″
f_c = the crushing strength of the rivets or plate on the projected area, say 24,000#/□″.
d = diameter of rivet, and t = thickness of plate.

The methods of failure are:

(1) Tearing of plate $= (p - d)\, t \cdot f_t$,

(2) Shearing of rivets $= \dfrac{\pi \cdot d^2}{4} \cdot f_s$, and

(3) Crushing $= t \cdot d \cdot f_c$.

Equating (2) and (3),

$$\frac{\pi \cdot d^2}{4} \cdot f_s = t \cdot d \cdot f_c \quad \text{or} \quad d = \frac{4\, t \cdot f_c}{\pi \cdot f_s}.$$

For the $\frac{5}{16}$″ plate $d = \dfrac{4 \times 5 \times 24{,}000}{\pi \times 16 \times 12{,}000} = 0.803″$.

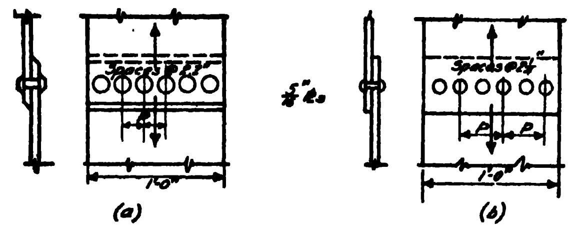


FIG. 33

This is not a commercial size. Use $\frac{7}{8}$″ rivets. If 0.80″ diameter rivets were used, equating either (2) or (3) to (1) would result in the same pitch in both cases and the three resistances would be equal. With a $\frac{7}{8}$″ diameter,

$$(2) \quad = \frac{\pi \times (\frac{7}{8})^2}{4} \times 12{,}000 = 7220\#.$$

$$(3) \quad = \tfrac{5}{16} \times \tfrac{7}{8} \times 24{,}000 = 6560\#.$$

Hence joint is controlled by bearing.

Equating (1) and (3),

$$t \cdot d \cdot f_c = (p - d)\, t \cdot f_t, \quad \text{or} \quad p = d + \frac{d \cdot f_c}{f_t}.$$

Substituting for this case,

$$p = 0.875 + \frac{0.875 \times 24{,}000}{16{,}000} = 2.2″.$$

Assuming the plate to tear,

$$\text{Efficiency} = \frac{(p - d)\, t \cdot f_t}{p \cdot t \cdot f_t} = \frac{p - d}{p} = \frac{2.2 - 1.0}{2.2} = 56\%.$$

In Fig. 430 (*b*), the structural joint, $\frac{3}{4}$″ rivets are arbitrarily used.

Bearing of $\frac{3}{4}$″ rivet on $\frac{5}{16}$″ plate = 5630#
Single shear = 5300#

5300# controls.
Rivets spaced 3 diameters as a minimum = $2\frac{1}{4}$″ o.c.
Net section between rivet centers

$$= (2\tfrac{1}{4} - \tfrac{7}{8}) \times 16{,}000 = 22{,}000\#$$
Strength = 5300#

In (*a*), the net section between rivet centers

$$= (2.2 - 1) \times 16{,}000 = 19{,}200\#$$
Strength = 6560#

$$\frac{5300}{22{,}000} = 0.24 \qquad \frac{6560}{19{,}200} = 0.34$$

Thus it is seen that (*a*) is more efficient than (*b*). The simplicity in (*b*) is an advantage and is the method used in structural joints.

In regard to the theory of riveting for structural work, the following discussion is taken with per-

mission from "Steel Structures."* The authors consider it to be one of the best treatises written on the subject.

"The following assumptions are made in designing riveted joints:

1. That all rivets completely fill the holes into which they are driven.
2. That the rivets in a compression member take the place of the metal punched out, but that in a tension member the section is weakened because the net section through the rivet hole is less than the gross section.
3. That a rivet cannot safely carry a tensile stress, that is, a stress pulling against its head.
4. That the friction between the parts joined should be neglected.
5. That the bending stress in the rivets may be neglected.
6. That the net section of a piece of steel will offer the same resistance per square inch as the gross section.
7. That the stress is equally distributed over the net section of the pieces joined in tension.
8. That the stress is equally distributed over all the rivets of a joint.

"As stated above, these assumptions are not rational, but nevertheless they are universally used in designing riveted connections because they give results which are consistent with the results of tests.

"These assumptions are largely interdependent and will be considered in detail.

"If a rivet were perfectly driven, and the hole completely filled when the rivet was hot, it would contract in diameter in cooling. This contraction precludes an intimate contact between the rivet and the walls of the hole.†

"Regardless of this fact it is the universal practice to proportion compression members for gross section, and tension members for net section. An allowance should, however, be made in compression members, for open holes, or holes for loose fitting bolts or pins.

"Coincident with the contraction in diameter while cooling, the length between heads tends to decrease, and a tensile stress is set up in the rivet. In addition to this stress, the metal which is being riveted together is compressed by the enormous pressure exerted by the riveting machine, and when this pressure is relieved, the metal tends to resume its unstrained form, and exerts a tensile stress on the rivet. This initial tension tends further to reduce the diameter of the cold rivet and cause a greater clearance between the rivet and the walls of the hole. The amount of the initial tensile stress on the rivet is a very uncertain quantity. It sometimes requires a very little pull on the head of a rivet to break it off. This is probably in part due to the heat treatment which it has received, making it nonhomogeneous. Nearly all specifications prohibit the use of rivets in direct tension, but they are nevertheless so used in certain connections, because the construction is usual and simple. In these connections there are usually stresses acting at right angles to each other, such as a shearing and a tensile stress. Bolts might be used to take the tension and rivets to take the shear, but rivets are generally used throughout.

"Experiments indicate that the clearance between the rivet and the walls of the hole allows a slip to take place when the friction between the parts is overcome.‡ Therefore friction is the resisting force in a riveted joint, so long as the stress is not great enough to produce slip. With good riveting and ordinary working stresses there is probably no slip,§ nevertheless rivets are calculated to resist shearing off. If a proper working stress is used, the shearing strength of a rivet is a proper measure of the friction produced, because the friction depends upon the tension in the rivet, and that, as well as the shearing strength, depends upon the area of the cross section. In good work the slip is so small that a joint may safely be strained beyond the slipping point, if the stresses do not alternate in direction.||

"Practically it is considered of great importance, that the rivets should completely fill the holes into which they are driven. Since this is impossible it is not of so much importance so long as sufficient friction is produced between the parts joined. As it requires great pressure to make a hot rivet fill the hole, especially when the holes in the parts joined do not come exactly opposite to each other, this pressure is useful in bringing the parts into intimate contact, which is necessary to develop the friction.

"If no slip occurs, the only bending stress in a rivet is due to elastic deformation, if any at all occurs. The longer the rivet the less the bending stress. Usually specifications require that the grip of a rivet shall not exceed from four to five times its diameter, on the supposition that the rivet transmits the stress. This requirement is necessary, because if the grip is great and the number of pieces to be riveted together is large, the pressure exerted by the riveting machine is not sufficient to bring the pieces into intimate contact and thus develop the friction.

"When rivet holes are punched, some of the material immediately surrounding the hole is injured, also a riveting machine exerts an

* By Prof. Clyde T. Morris — Copyright by the McGraw-Hill Book Co., Inc.

† According to experiments by M. Considere in 1886, the space between the rivet and the side of the hole varies from 0.002 to 0.02 inch. See Bulletin No. 62 American Railway Engineering Association, page 149.

‡ Contraction of rivets in cooling is much greater than their elastic strength.

$$E = 30{,}000{,}000\#/\square'' \quad \text{E.L. in tension} = 30{,}000\#/\square''$$

$$E = \frac{P}{A} + \frac{e}{l} \qquad \frac{e}{l} = \frac{30{,}000}{30{,}000{,}000} = .001 \text{ length — elastic stretch}$$

coefficient = 0.0000065/°F.

$$\frac{.001}{.0000065} = 154^\circ \text{ F. change in temperature to bring rivets to their E.L.}$$

if they were not allowed to contract.

Therefore the rivets hold the plates tightly together.

§ Experiments indicate that slip occurs at a stress of from 11,500 lbs. to 21,900 lbs. per sq. in. of rivet cross-section.

|| See Bulletin No. 62 Am. Ry. Eng. Assoc., pages 3 and 4.

enormous pressure on the metal near the rivet, and may overstrain it. These might tend to reduce the permissible unit stress in tension on the net section,* but experiments show that where the section is suddenly reduced, as in a notched bar or in a section through rivet holes, the ultimate strength per square inch is increased by an amount which will more than equal the reduction due to injury.†

"If then the distribution of stress over the net section through the rivet holes is uniform, as per the 7th assumption, there is no reason why the allowable intensity of stress should not be as great as for a section without rivet holes. If, however, the stress is unequally distributed, the maximum intensity will be greater than the 7th assumption will give.

"There are a number of causes producing non-uniform distribution of stress over the net section through rivet holes. If two plates in tension be joined by several rows of rivets, and there is no slip, the stress is transmitted from one to the other by means of the friction at their surfaces of contact. This friction is greatest under the rivet heads, because the friction is produced by the tension in the rivets. Therefore the intensity of stress is greater under the rivet heads than half way between them. If the stress is tensile in the plates joined, the uniform distribution of stress will be interfered with, as in a notched bar.†

"The result is, no doubt, a somewhat greater intensity of stress near the rivet holes than half way between them.

"If the stress is not equal on all the rivets in a cross section, as per the 8th assumption, there may be a large variation in intensity of stress over the section. On this account the rivets in a joint should be symmetrically disposed about the center lines of stress, and eccentric stresses avoided wherever possible. If any of the rivets are defective, the result may be the same as that of an unsymmetrical distribution.

"If the friction which is produced by the rivets is greatest under the rivet heads, the stress is transferred from one plate to the other in a series of increments. The stress in one plate increases, while that in the other decreases. The result is that the intensity of stress in the two plates at a cross section is not equal, and this tends to cause one plate to deform more than the other and thus throw more stress on the rivets at one end of the joint in one plate and upon those at the other end in the other plate. But the plates cannot deform unequally as long as there is no slip, so there is no reason why there should not be a uniform distribution of stress over the rivets, as long as they are all in the same condition. This would require perfect workmanship."

20. Requirements for Good Riveted Joints.

From the discussion above it is seen that: Good riveted joints,

"1. Should be as compact as possible, in order to render the uniform distribution of stress more certain.

2. Should not be very large, because the workmanship cannot be perfect, and there is the greatest danger of uneven distribution of stress in a joint having the largest number of rivets. With part of the rivets in a joint defective there may be eccentric stresses and overstrain, causing a redistribution of stress and probably overstrain in other members.

3. Should have its rivets arranged symmetrically about the center lines of stress.

4. Should have provision for unavoidable eccentric stresses.

5. Should have rivets of good material, properly driven, under uniform conditions.

6. Should have a sufficient number of rivets so that there will be no slip if the stresses alternate in direction.

7. Should not have rivets in direct tension."‡

21. Ways of Failure.

A riveted joint may fail in one or more of the following ways:

(1) by single shear,
(2) by double shear,
(3) by tearing out at the edges, or breaking through a hole,
(4) by bearing of the metal on the rivet, or
(5) by tension through the net section.

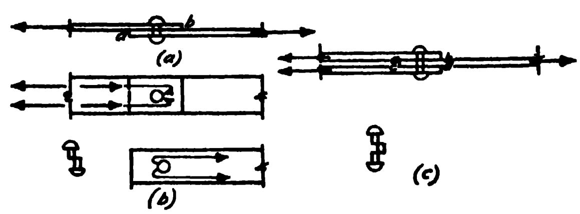


Fig. 34

Single shear may be defined as occurring when all the parts which tend to move in one direction are on the same side of and adjacent to the parts which tend to move in the opposite direction. In Fig. 34 (*a*), there is a tendency of one plate to slide on the other at the plane *a–b*, causing one section of the rivet to tend to shear on the other; (*b*) traces the lines of stress. If the friction of one plate on the other is neglected, the area resisting the pull shown is the cross-section of the rivet. The nominal diameter should be used, for if there were any misalignment of the parts, the nominal section might be the least. Figure 35 (*d*) shows a typical single shear failure.

* See Proceedings of the Inst. of Mech. Eng., August, 1887, page 326.

† See Proceedings of the Inst. of Mech. Eng., October, 1888; also see Heller's "Stresses in Structures".

‡ From "Steel Structures" by Prof. Clyde T. Morris — Copyright by the McGraw-Hill Book Co., Inc.

Double shear may be defined as occurring when the parts which tend to move in one direction are between the parts which tend to move in the op-

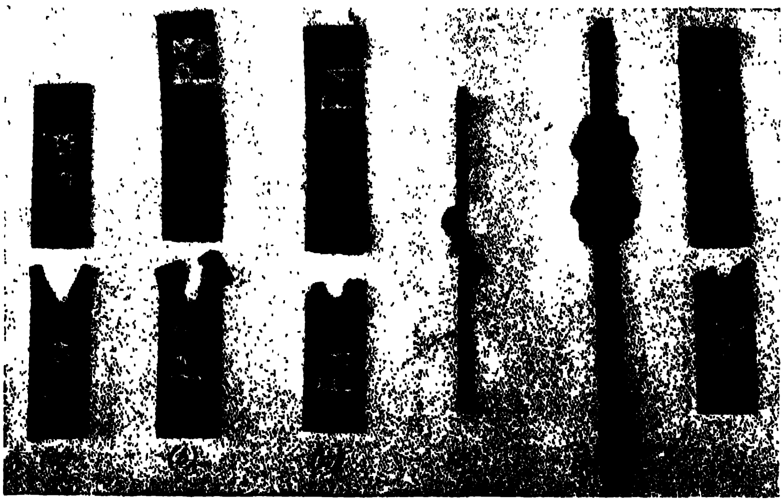


FIG. 35

posite direction. In Fig. 34 (*c*), there is a tendency of the two outside plates to slide on the middle one at the planes *a–b* and *c–d*, causing a section of the rivet to slide on the remainder at two places. Before the joint can rupture, both sections of the rivet must fail. The lines of stress may be traced similarly to those shown in (*b*). Obviously, the strength of a rivet in double shear is twice that of the same rivet in single shear. Single shear and double shear could not both act in any one given case. Figure 35 (*e*) shows a typical failure.

Tearing out at the edges of a joint is prevented by making the edge distance sufficiently large. The usual assumed minimum edge distances have proven to be relatively satisfactory as determined by practice and in experiments (Art. 25). The failure in (*a*), Fig. 36, is more likely to occur than that in (*b*), but

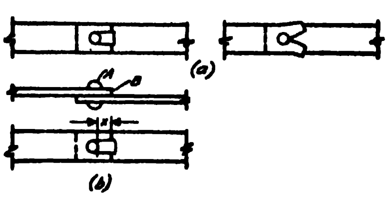


FIG. 36

the shear resisting areas in any case, provided by the distance, x, are uncertain. The tension passes AB as shear, and the compression against the rivet passes to the lower plate by shear. At the point where the rivet bears against the plate, compression, or in reality, one form of bearing, results. Figure 35 (*a*) shows a typical failure of this kind. Figure 37 shows a typical multiple punch, such as is used to punch the holes for the riveting.

When the diameter of the rivet is large in proportion to the thickness of the plates connected, the rivet has a tendency to cut into the plates. This is called a **bearing** failure. The resistance depends upon the area under pressure, or upon the thickness of the plate. The rivet itself might also crush. The forces in Fig. 38 (*c*) are not all vertical, but instead they are nearly normal to the circumference of the rivet. The resultant pressure in a given direction would be the same as if the unit pressure were exerted upon the projection, d, of the circumference. The crushing of the plate by the rivet may be considered the same as the crushing of the rivet by the plate, so that the resistance of the plate is its thickness times the diameter of the rivet times the allowable stress. The resisting area is then the projection on a plane parallel to the axis of the rivet, of a semicircumference in contact with the plate times the

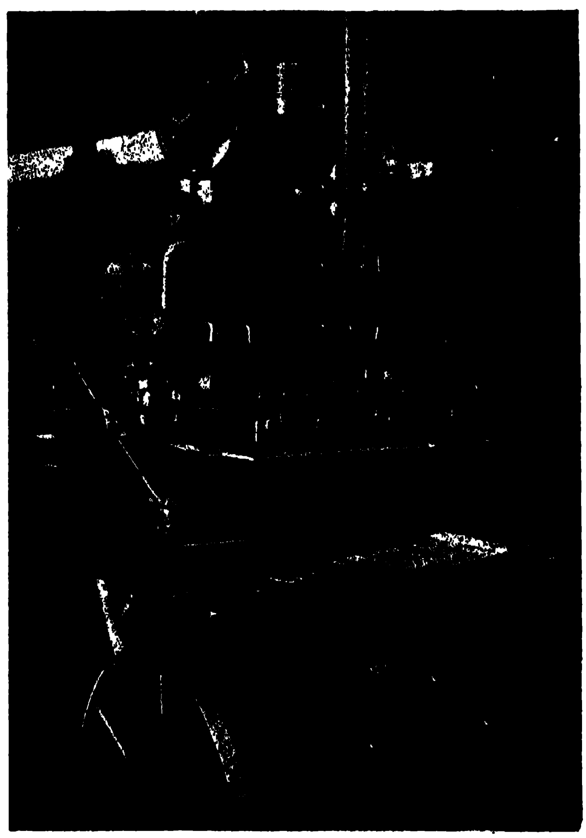


FIG. 37*

* Courtesy of New England Structural Co.

thickness of the plate. The controlling value is established by the least combined thickness of plates which act in a single direction. In Fig. 39 (*a*), if P_1 and P_2 balance with respect to the center of the hole, the bearing is $P_1 + P_2$, uniformly distributed over the area, A. If P_1 and P_2 are not balanced, as in (*b*), the average pressure is the same but the

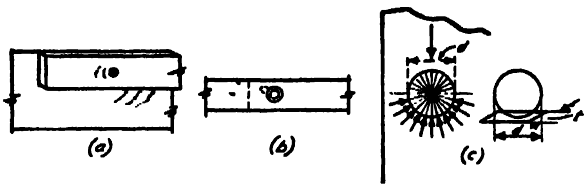


FIG. 38

maximum stress is greater and is on the side of the larger lever arm. If there is a force on only one side of the plate, as in (*c*), the smaller the distance between the force and a parallel axis of the plate, the smaller the maximum bearing stress. The stresses are however assumed to be uniformly distributed, in practice, to avoid involved calculations. Excessive crushing

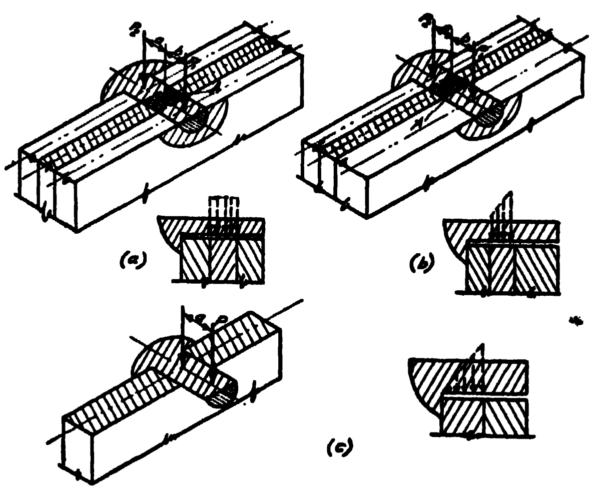


FIG. 39

eventually results in the shearing of the rivets unless the plate tears out in front of the rivets. For this reason it is often difficult to discriminate between crushing and shearing failures. Figure 35 (*b*) shows a typical bearing failure. Figure 40 shows a typical gang drill. This is used for thick metal when the holes cannot be punched.

When a hole is punched through the resisting section, the plate is naturally reduced in cross-sectional area, and the remaining **net section** may govern the **tensile resistance** of the joint. The rivet does not prevent the joint in Fig. 41 (*a*) from separating, if the plate breaks. The cross-sectional area of a member after the holes are deducted is called the net section. The holes are always punched $\frac{1}{16}''$ greater than the diameter of the rivet, in shop practice. In the process of punching, the metal immediately surrounding the hole is probably stressed beyond its elastic limit so that the universal

FIG. 40*

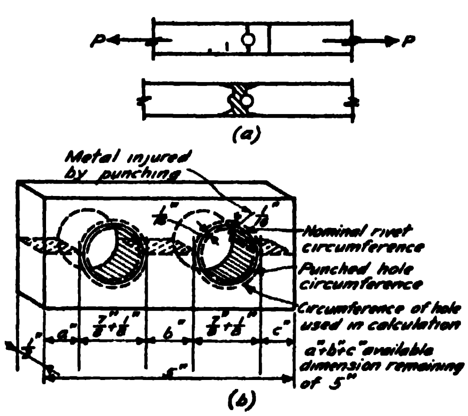


FIG. 41

* Courtesy of New England Structural Co.

custom is to add $\frac{1}{16}''$ more to the diameter of the hole to use as the basic dimension, and not to count upon this overstressed metal.* The diameter used in the calculation to reduce the gross section for each rivet is then the nominal diameter plus $\frac{1}{8}''$.† The net section of a $5'' \times \frac{1}{2}''$ plate with 2-$\frac{7}{8}''$ rivets in a transverse line is, for instance, $5 \times \frac{1}{2} - 2 (1 \times \frac{1}{2}) = 1.50\square''$ (the sum of the shaded areas in Fig. 41 (*b*)). Figure 35 (*c*) and (*f*) shows typical tension failures. Figure 42 shows an electric heater for reheating the rivets before they are driven.

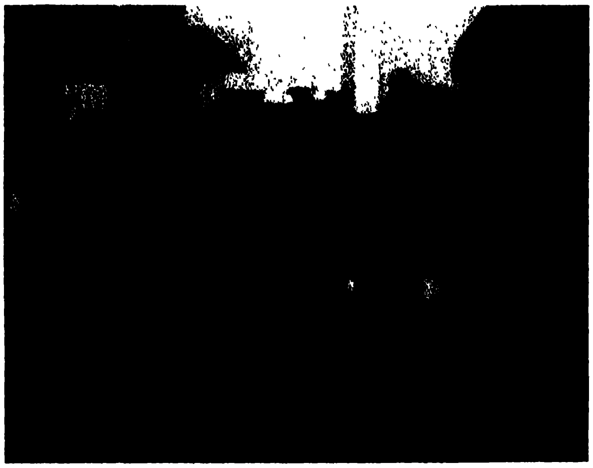


FIG. 42‡

The action involved in the foregoing ways of joint failures may be well illustrated by making cardboard specimens with matches for rivets, as suggested in Fig. 43.

Prob. 21a. What are the net sections in (*a*) and (*b*), Fig. 44?

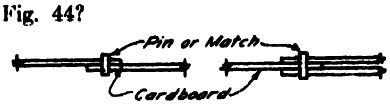


FIG. 43

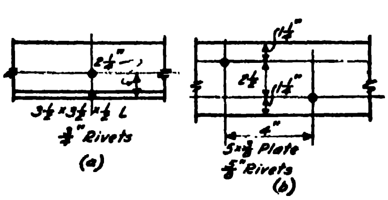


FIG. 44

which also cuts off the rod at a desired length to make the shank. In order to make a rivet effective in holding the parts together, a second head is formed by a riveter when the parts are in place. Figure 45 shows a large pneumatic riveter. Figure 46 (*b*) shows the standard proportions of button-head and countersunk-head rivets. The button

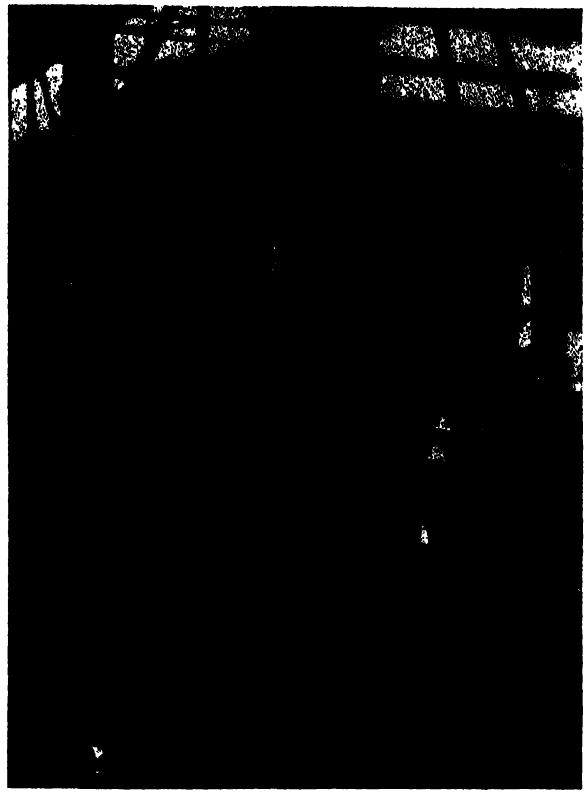


FIG. 45‡

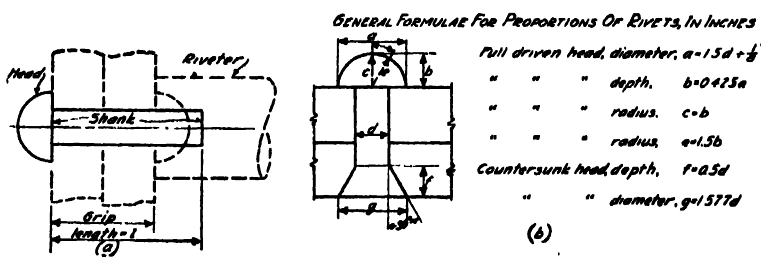


FIG. 46

22. General Proportions of Rivets.

Rivets are made in a machine which upsets one end of a heated, soft steel rod to make a head, and head may be flattened for purposes of clearance if desired. There are many types of heads for rivets used in other kinds of work, but the three forms just mentioned are the ones ordinarily used for structural steel. The full head is used in all cases except where clearance (flattened head) or a smooth surface (countersunk head) is required. A countersunk head should not be used if the thick-

* In Haven and Swett's treatise, on page 92, it is pointed out that the loss due to punching is about 12%. In Johnson's "Materials of Construction" — John Wiley & Sons, Inc. — it is stated that the injurious effect resulting from punching the hole is principally on the compression side.

† The reduction for the net section for a countersunk rivet (Fig. 46) should be based upon a diameter $\frac{1}{4}''$ greater than the rivet diameter when the thickness of the plate is $\frac{1}{2}''$ or less.

‡ Courtesy of New England Structural Co.

ness of plate involved is less than one-half the diameter of rivet (Fig. 46 (*b*)), and should be avoided whenever possible.

The rivet is described by stating its diameter, d, and the length, l, under the head, as $\frac{3}{4}'' \times 4\frac{1}{2}''$. The grip is the distance between the finished heads. This distance should be figured as slightly more than the sum of the thicknesses of the parts to be held together on account of the roughness of the surfaces in contact, an allowance of $\frac{1}{32}''$ being made for each surface. From this grip the length required under the head* may be calculated by first increasing the grip in the ratio of the area of the hole to be filled to the nominal area of the rivet section, and then adding to this value a length to make the head. The last value amounts to the length of a cylinder, of the rivet diameter, which is equal in volume to that of a head.

Illustrative Prob. 22a. What length of $\frac{3}{4}''$ rivets are required to fasten 3-$\frac{1}{2}''$ plates together?

$\Sigma t = \frac{1}{2} + \frac{1}{2} + \frac{1}{2} = 1\frac{1}{2}''$ 4 surfaces of contact $4 \times \frac{1}{32} = \frac{1}{8}''$.

Grip $= 1\frac{1}{2} + \frac{1}{8} = 1\frac{5}{8}''$.

$$\frac{\text{Area of hole}}{\text{Area of rivet}} = \frac{\frac{\pi\,(\frac{13}{16})^2}{4}}{\frac{\pi\,(\frac{3}{4})^2}{4}} = \frac{169}{144}. \qquad \text{Adjusted grip} = 1\tfrac{5}{8} \times \frac{169}{144} = 1\tfrac{29}{32}''.$$

Refer to Fig. 46 (*b*). $a = 1.5 \times \frac{3}{4} + \frac{1}{8} = 1\frac{1}{4}''$

$b = 0.425\,(1\frac{1}{4}) = 0.532''$.

$$\therefore \quad \text{Volume of head} = \frac{\pi\, b}{24}\,(3\,a^2 + 4\,b^2) = 0.40 \text{ cu. in.}$$

$$\text{Equivalent length of rivet} = c\,\frac{\pi\,(\frac{3}{4})^2}{4} = 0.40 \qquad c = 0.96'' \text{ say } 1''.$$

$$\text{Length of rivet} = 1\tfrac{29}{32}'' + 1'' = 2\tfrac{29}{32}''.$$

Lengths are given to the nearest $\frac{1}{4}''$. Use 3″.

The maximum grip should preferably not be more than four diameters of the rivet, so that excessive bending stresses may not occur in the rivets.

Prob. 22b. Calculate the length of a $\frac{7}{8}''$ rivet to supply a grip of $2\frac{1}{2}''$. Check your result by structural tables.

23. Common Sizes of Rivets Used.

In structural work, $\frac{5}{8}''$, $\frac{3}{4}''$ and $\frac{7}{8}''$ are the sizes of rivets commonly employed, while 1″ rivets are used occasionally for heavy work. Smaller rivets are used for ornamental iron work principally, and seldom for structural work. The majority of rivets used are $\frac{3}{4}''$, with $\frac{5}{8}''$ for special, light framing and $\frac{7}{8}''$ for large plate girders and trusses. It is difficult to drive rivets larger than $\frac{7}{8}''$ with the hand riveting hammer and it is not always possible to obtain the necessary power to drive them satisfactorily by machine. It is a fairly safe rule to use the largest sized rivet possible, but the number of sizes used should be limited to not over two for a job, and when economy is best served, to one size. Any individual, heavy member should have only one size of rivet in it in order to avoid extra handling. A common rule is that the rivet should not have a diameter greater than the thickness of the thickest metal connected:† Somewhat thicker metal than this can be punched, and some companies have no trouble in punching a plate $\frac{1}{8}''$ greater in thickness than the diameter of the rivet, as a limit. This is apt to be expensive as punches become dull and thus are often broken, and the thicker the plate, the more the injury to the plate and the punch. As a general rule, $\frac{7}{8}''$ metal should be the maximum used, and holes through thicker metal should be drilled. The maximum size of rivets for the flanges or legs of the usual structural shapes are given in Table 21. These are determined on the basis of

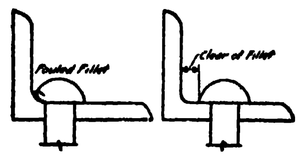


FIG. 47

TABLE 21

MAXIMUM SIZES OF RIVETS

Standard Beams		Structural Channels	
18″ and deeper	$\frac{7}{8}$	12″ and deeper	$\frac{7}{8}$
8″ to 15″ incl.	$\frac{3}{4}$	8″, 9″ and 10″	$\frac{3}{4}$
6″ and 7″	$\frac{5}{8}$‡	6″ and 7″	$\frac{5}{8}$‡
4″ and 5″	$\frac{1}{2}$	3″, 4″ and 5″	$\frac{1}{2}$
3″	$\frac{3}{8}$		
Bethlehem Girder Beams		**Bethlehem I Beams**	
12″ to 30″ incl.	1	30″, 28″ and 26″	1
8″, 9″ and 10″	$\frac{7}{8}$	15″ to 24″ incl.	$\frac{7}{8}$
		8″ to 12″ incl.	$\frac{3}{4}$

GAUGES FOR ANGLES, INCHES (FIG. 87)

Leg	8	7	6	5	4	$3\frac{1}{2}$	3	$2\frac{1}{2}$	2	$1\frac{3}{4}$	$1\frac{1}{2}$	$1\frac{3}{8}$	$1\frac{1}{4}$	1	$\frac{3}{4}$
g_1	$4\frac{1}{2}$	4	$3\frac{1}{2}$	3	$2\frac{1}{2}$	2	$1\frac{3}{4}$	$1\frac{3}{8}$	$1\frac{1}{8}$	1	$\frac{7}{8}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{1}{2}$
g_2	3	$2\frac{1}{2}$	$2\frac{1}{2}$	2					‡						
g_3	3	3	$2\frac{1}{4}$	$1\frac{3}{4}$											
Max. rivet	$1\frac{1}{8}$	1	$\frac{7}{8}$	$\frac{7}{8}$	$\frac{7}{8}$	$\frac{7}{8}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{1}{4}$

g_1 is for only one line of rivets in a leg.

g_2 and g_3 are for two lines of rivets in a leg.

For column details 6″ leg ($\frac{1}{2}$ inch thick or less) against column shaft, $g_2 = 1\frac{3}{4}''$, $g_3 = 3''$.

For diagonal angles, etc., gage in middle, where riveted leg equals or exceeds 3″ for $\frac{3}{4}''$ rivets, $3\frac{1}{2}''$ for $\frac{7}{8}''$ rivets.

Use special gages to adapt work to multiple punch, or to secure desirable details.

* Lengths of rivets required for various grips may be found in the "Pocket Companion," Carnegie Steel Co.

† The diameter of a rivet also should not be greater than one-fourth of the width of the member connected.

‡ It should now be more evident that beams and channels less than 8″ deep, and angles with legs less than $2\frac{1}{2}''$ wide are not commonly used in important structural steel framing where holes in the flanges or legs are required, because rivets less than $\frac{5}{8}''$ would have to be used. This complicates the punching work. See Art. 3.

heading the rivet without materially engaging the fillet and maintaining the standard gauges and edge distances (Fig. 47).

Prob. 23a. What is the maximum sized rivet allowable in the flange of a 12″ beam? In the 4″ leg of an angle? In the flange of a 10″ channel?

24. Bolts for Structural Steel Work.

Bolts are sometimes used in place of rivets for minor field connections, as discussed in Art. 18. The ways of failure in joints are the same as has been discussed (Art. 21), whether rivets or bolts are used. The sizes of bolts commonly used are the same as those of rivets (Art. 23), and the same requirements for their spacing is always specified (Art. 25). The general proportions of bolts are discussed in Art. 13, (Book I) and the theory of the stresses induced by the external forces has also been considered therewith.

25. Requirements for Locating Rivets and Bolts.

The dimensions for locating rivets are determined almost universally by practical rules which are the result of tests. If the rivets are spaced too closely together the metal between them would be of little value in tension as it would tend to tear as illustrated in Fig. 48 (*a*). If the rivets are too far apart, the parts might tend to buckle when subjected to compression, as shown in (*b*), and hence develop high local

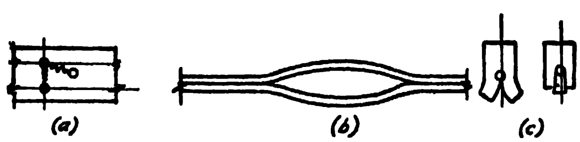


FIG. 48

stresses. Metal should also be held well together so that dirt, water and rust will not impair the joint. If rivets are too near the ends or edges of a plate, they will tend to tear out or split the plate, as shown in (*c*). The distance center to center of rivets parallel to the line of stress, whether in the same or different rows, is called the **pitch.** The spacing of rivet lines in a direction which is transverse to the line of stress is called the **gauge.**

SPECIFICATION CLAUSES

Spacing Center to Center The minimum spacing shall not be less than three diameters of the rivet.

The maximum spacing in a direction with the stress shall not exceed 16 times the thickness of the thinnest metal, or 6″ as a maximum.

The maximum spacing in a direction perpendicular to the line of stress shall not exceed twice the pitch parallel to the direction of stress (or a maximum of 40 times the thickness of the thinnest plate*).

* Alternate wording.

The minimum spacing is seldom used in practice, and a spacing a little larger is employed, to avoid close rivets. For instance, for $\frac{3}{4}''$ rivets,

$3 \times \frac{3}{4} = 2\frac{1}{4}''$ minimum: — usual minimum $2\frac{1}{2}''$.

The minimum pitch in a double line can be less than that in a single line, but the center to center of holes in any direction cannot be less than the minimum. The specification for the maximum spacing was established "during the early days of iron fabrication when such a provision may have been needed but that is not needed now. . . . It is recommended to designers of steel structures that the precedent established by two or three generations of specification writers be cast aside and that the maximum spacing for rivets be increased."† Some structural companies have made a start in this direction and allow a maximum of 8″ for $\frac{7}{8}''$ ϕ rivets in many cases. The specification for the maximum spacing in a direction perpendicular to the line of stress is seldom used except in connection with cover plates of built-up members. Some engineers specify that when plates are in contact, rivets should not be farther apart than 12″ o.c. in either direction to hold the plates well together.

SPECIFICATION CLAUSE

Edge Distance The minimum edge distance shall not be less than one and one-half times the diameter of rivet.

The maximum edge distance shall not be greater than 8 times the thickness of the thinnest metal.

Again, the minimum edge distance is not usually used in practice, especially when the edges are sheared. For instance, for $\frac{3}{4}''$ ϕ rivets, $1\frac{1}{2} \times \frac{3}{4} = 1\frac{1}{8}''$ minimum; usual minimum $1\frac{1}{4}''$. The specification for maximum edge distance is given to avoid any tendency of the plate to "curling" about the rivet.

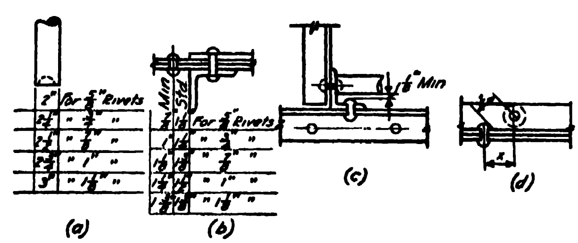


FIG. 49

The center of a rivet must be placed far enough away from any projection so that the rivet can be headed properly. Figure 49 (*a*) shows standard dimensions of rivet dies, and **clearances** are based on these. Such clearances are shown in Fig. 49 (*b*). The standard is obtained by adding $\frac{1}{8}''$ to one-

† From an article by R. Fleming in Engineering News Record, June 3, 1920.

half the die diameter. The minimum is $\frac{1}{8}''$ less than this diameter. The latter clearance should be used only when it is absolutely necessary, as the die has to be tipped slightly in order to head the rivet. Some fabricators have a special riveting tool which has one-half of the head cut off. The clearance for such a die might be reduced to one-half the diameter of the rivet head plus $\frac{1}{8}''$. It is not good practice to call for such work, however. If it is desired to drive a rivet in a plane at right angles to another containing rivets as in Fig. 49 (c), a clearance of $\frac{1}{8}''$ over the height of the head of the existing rivet must be maintained for the die. The head of the existing rivet may be high on account of excessive length before driving. Gauges in opposite legs of angles, as shown in Table 21, are based upon this reasoning.

TABLE 22

RIVET SPACING

Diam. of Rivet	Center to Center				Edge Distance	
	Min.	Preferable Min.	Preferable Max.	Max.	Sheared Edge	Rolled or Planed Edge
$\frac{7}{8}$	$2\frac{5}{8}$	3	6	6	$1\frac{1}{2}$	$1\frac{1}{4}$
$\frac{3}{4}$	$2\frac{1}{4}$	$2\frac{1}{2}$	6	6	$1\frac{1}{4}$	$1\frac{1}{8}$
$\frac{5}{8}$	$1\frac{7}{8}$	2	$4\frac{1}{2}$	6	$1\frac{1}{8}$	1
$\frac{1}{2}$	2	$1\frac{3}{4}$	4	6	1	$\frac{7}{8}$

multiple punches which punch several holes at once (Fig. 37). They are used in connection with a spacing table so that there are certain limitations, such as punching certain parts at once. No template is used.

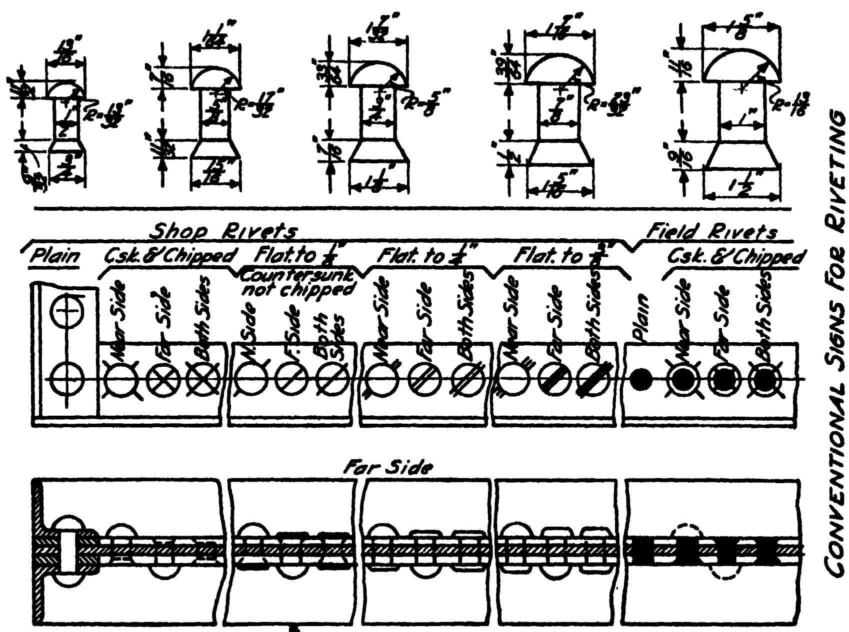


Fig. 50

Illustrative Prob. 25a. Calculate a theoretical gauge for a $4 \times 4 \times \frac{1}{2}$ L.

From Table 21, the maximum size of rivets is $\frac{7}{8}''$ ϕ.
The height of the head is approximately $\frac{5}{8}''$ (see Fig. 46).
The diameter of the die is $2\frac{1}{2}''$
Referring to Fig. 49 (c),

$$\tfrac{1}{2} + \tfrac{5}{8} + \tfrac{1}{8} + \tfrac{1}{2}(2\tfrac{1}{2}) = 2\tfrac{1}{2}'', \text{ the gauge of the angle.}$$

In a similar manner the distance, z, Fig. 49 (*d*), which must be equal to one-half the standard die dimension at least, controls the spacing, x, there shown. The controlling dimensions for rivet spacings and edge distances are summarized in Table 22.

Modern fabrication plants now have installed

The spacing of rivets may be arranged to suit the spacing table in a great many instances by keeping symmetrical arrangements as far as possible.

26. Conventional Signs for Rivets.

It is important for the structural engineer to be familiar with the conventional signs for rivets, for although he may not be called upon to make or design the details, he may have to check them when they are submitted for approval. He certainly should not select members in which there is insufficient clearance for the necessary rivets. Figure 50 shows the conventional signs for indicating rivets on structural drawings.* The flattened heads shown, and

* Sometimes called the Osborne system.

used for clearance, are standardized as $\frac{1}{8}''$, $\frac{1}{4}''$ and $\frac{3}{8}''$ high. When rivet heads are required to be only $\frac{1}{8}''$ high or less, they are made by countersinking and not chipping. If one part requires a smooth surface the rivets affected have to be countersunk and chipped. Not all rivets are driven in the shop (shop rivets), and some are usually left to be driven during the erection (field rivets). The symbols distinguish the two kinds.

The usual office practice is to show the centers of the rivets and the heads only (drawn to an approximate scale) but seldom showing the rivets in section, except to determine a clearance, and then perhaps only in pencil. Some offices merely indicate the centers of the rivets by crosses, while others show the end and significant rivets, merely indicating that so many should go in between at a certain pitch, all tied to definite working points.

Prob. 26a. Make a sketch of a $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ angle 4′-0″ long. Show the gauge line in elevation. What is the gauge? Show a number of rivets 3″ o.c. Indicate the first countersunk near side, the second countersunk far side, the third flattened to $\frac{1}{4}''$ near side.

27. Allowable Stresses.

There is no very definite relation between the working stresses involved in riveted joints, as they are based upon the results of experiments. Since the tensile strength of steel is more positively known than the compressive strength, the former is usually made the standard of comparison. The specified **unit tension** is **16,000#/□″**, and this is the allowable for the net section. The **unit shearing stress** for shop rivets is often specified as three-quarters of the unit tension, or $\frac{3}{4} \times 16{,}000 =$ **12,000#/□″**, although many codes allow only 10,000#/□″. Experiments* have shown that the **bearing strength** of a rivet is about 1.8 times its shearing strength, so that on this basis, it is often taken as **twice** the value allowed for **single shear.** For a 12,000#/□″ value of shear, the allowable bearing is thus 24,000 #/□″; for 10,000#/□″ shear, 20,000#/□″.

When a rivet bears upon metal which is supported in a sidewise direction by adjacent metal, the bearing value of the rivet is probably somewhat larger due to the increased friction and grip. In Fig. 51, the rivet shown bearing on the web of the I beam can be counted upon for additional bearing resistance, for the metal below the rivet is restrained by relatively stiff connection angles each side of it. If the metal is to crush, it must spread sidewise, but the restraint tends to prevent this action. Such resistance is called **enclosed bearing.** Some specifications allow the usual bearing stress of 24,000#/□″ (unenclosed) to be increased to **30,000#/□″** for such cases.† This increased allowance should, however, be used with discretion; that is, a plate between two other plates is not restrained in the same sense that a plate between two angles or channels is.

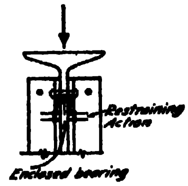


FIG. 51

The allowable stresses for **field rivets** and turned bolts (those which are turned in a lathe to an exact fit) are generally reduced because the frictional resistance of such rivets is often less, and in a great many cases the rivets are not driven under as favorable conditions as shop rivets are. A 20% reduction in strength has been shown to be the average in experiments. Referring to the unit stress for shop rivets,

$$12{,}000 \times 0.8 = 9600\#/\square''.$$

A common specification is 10,000#/□″ for single shear and 20,000#/□″ for bearing.

Rough bolts (the kind usually employed either in the shop or the field) are generally not of as good material as rivets, and are made from rods in an automatic machine and experiments have shown their strength to be approximately two-thirds that of shop rivets. Thus

$$12{,}000 \times \frac{2}{3} = 8000\#/\square''.$$

Many codes specify 8000#/□″ for single shear, and 16,000#/□″ for bearing. A feature in connection with the bearing of bolts on steel which is too often neglected is that of the effect of the threads. A bolt is often threaded for a considerable distance

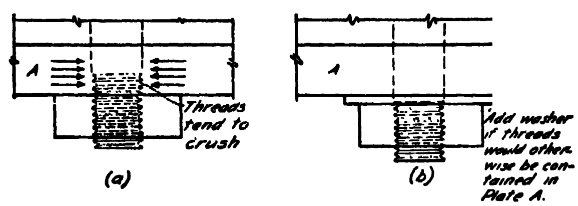


FIG. 52

at its end. If any of the threaded portion of the bolt is in contact with the metal to be connected, the bearing resistance is often seriously impaired because the relatively sharp threads will be easily crushed.

This is illustrated in Fig. 52 (*a*). To relieve this action, a washer should be used, as shown in (*b*), if the threaded portion of the bolt is too long.

Building codes vary to some extent in their specifications for the allowable stresses to be used in the

* See Johnson's "Materials of Construction" — John Wiley & Sons, Inc.

† The late Mr. R. H. Brown, formerly chief engineer of the Eastern Bridge and Structural Co., Worcester, Mass., was a strong advocate that such an allowance is not unreasonable. The authors have made a number of tests which verify this reasoning. In double shear, there is a tendency toward a beam action of a concentrated load between two supports ($M = P \cdot l \div 4$), whereas in single shear, there is a tendency toward cantilever action of a half span ($M = P \cdot \frac{l}{2} \div 2$). The latter seems more positive. See assumption 4, page 40.

design of joints. In any particular instance the ruling code must be followed. Table 23 summarizes the preceding discussion and the values given are recommended for use unless otherwise specified.

TABLE 23

ALLOWABLE UNIT STRESSES FOR RIVETS (#/□")

Shear	Rivets.........Shop 12,000	Bearing	Rivets, enclosed..Shop 30,000
	Rivets.........Field 10,000		Rivets, one side...Shop 24,000
	Turned Bolts..Field 10,000		Rivets and Turned Bolts...Field 20,000
	Rough Bolts...Field 8,000		Rough Bolts......Field 16,000

The strength of the individual rivets is calculated on the basis of nominal diameters because the nominal section may be the least, in case there is any misalignment of parts. No reduction is usually made for heads flattened to not less than $\frac{3}{8}$", since rivets are counted upon to take no axial tension.* The strength of countersunk rivets varies according to the proportion of the countersinking. If the metal below the countersunk portion is thick enough to develop the single shearing strength of a rivet by bearing, no reduction need be made. In order to avoid investigating minute details, the rule usually

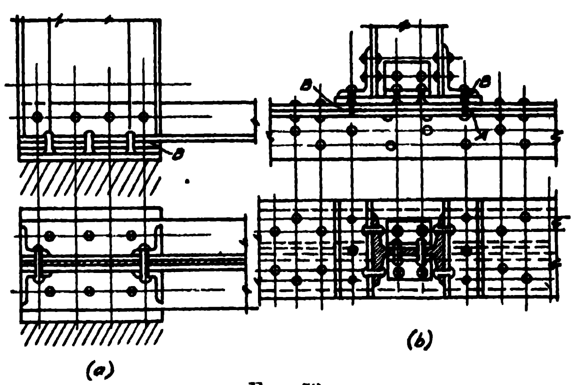


Fig. 53

followed is to allow no value for countersunk rivets if the thickness of the remaining metal is less than one-half the diameter of the rivet. Rivets flattened to less than $\frac{3}{8}$" and countersunk rivets often occur where they do not carry any considerable lateral force, as at point "*B*" in Fig. 53 (*a*). In general, designers do not pay much attention to these details, but in special instances, such as at *A* in Fig. 53 (*b*), the investigation is too often neglected.

The allowable value for an ordinary rivet is then the product of the allowable stress and the resisting cross-section. Thus the safe resistance of a $\frac{3}{4}$" shop rivet to single shear is

$$r = \frac{\pi \cdot d^2}{4} \cdot f_v = \frac{\pi \times (\frac{3}{4})^2}{4} = 0.4414 \times 12{,}000$$
$$= 5300\#.$$

Similarly, the safe resistance of a $\frac{3}{4}$" shop rivet (unenclosed) to bearing on a $\frac{1}{2}$" plate is

$$r = d \cdot t \cdot f_b = \frac{3}{4} \times \frac{1}{2} \times 24{,}000 = 9000\#, \text{ and so on.}$$

Rivets are practically never investigated for induced bending stresses. When the grip exceeds four diameters of the rivet, however, the bending should be considered.† In order to again avoid investigating minute details, the rule usually followed is to add 1% to the number of rivets otherwise required for each $\frac{1}{16}$" increase in a grip over four diameters of the rivet. Table 24 gives the shearing and bearing values for different sizes of rivets.

TABLE 24

RIVETS

Shearing and Bearing Values

Values in Pounds, Dimensions in Inches

Three-quarter Inch Rivets — Area .4418 Square Inch

Shear	Unit, Lbs. per Sq. In.	7000	8000	9000	10000	11000	12000	Enclosed Bearing
	Single Shear per Rivet	3090	3530	3980	4420	4860	5300	
	Double Shear per Rivet	6190	7070	7950	8840	9720	10600	
Bearing	Unit, Lbs. per Sq. In.	14000	16000	18000	20000	22000	24000	30000
Thickness in Inches	$\frac{1}{4}$	2630	3000	3380	3750	4130	4500	5630
	$\frac{5}{16}$	3280	3750	4220	4690	5160	5630	7040
	$\frac{3}{8}$	3940	4500	5060	5630	6190	6750	8440
	$\frac{7}{16}$	4590	5250	5910	6560	7220	7880	11250
	$\frac{1}{2}$	5250	6000	6750	7500	8250	9000	......
	$\frac{9}{16}$	5910	6750	7590	8440	9280	10130	......
	$\frac{5}{8}$	6560	7500	8440	9380	10310	11250	......

Seven-eighths Inch Rivets — Area .6013 Square Inch

Shear	Unit, Lbs. per Sq. In.	7000	8000	9000	10000	11000	12000	Enclosed Bearing
	Single Shear per Rivet	4210	4810	5410	6010	6610	7220	
	Double Shear per Rivet	8420	9620	10820	12030	13230	14430	
Bearing	Unit, Lbs. per Sq. In.	14000	16000	18000	20000	22000	24000	30000
Thickness in Inches	$\frac{1}{4}$	3060	3500	3940	4380	4810	5250	6560
	$\frac{5}{16}$	3830	4380	4920	5470	6020	6560	8200
	$\frac{3}{8}$	4590	5250	5910	6560	7220	7880	10840
	$\frac{7}{16}$	5360	6130	6890	7660	8420	9190	11500
	$\frac{1}{2}$	6130	7000	7880	8750	9630	10500	13130
	$\frac{9}{16}$	6890	7880	8860	9840	10830	11810	......
	$\frac{5}{8}$	7660	8750	9840	10940	12030	13130	......
	$\frac{11}{16}$	8420	9630	10830	12030	13230	14430	....

Values above upper dotted lines are less than single shear.
Values below lower dotted lines are greater than double shear.

Prob. 27a. Check the single shearing strength of a $\frac{7}{8}$" rivet if the unit stress is 12,000#/□". What is the value of the rivet as controlled by bearing on a $\frac{3}{8}$" plate if the unit stress is 24,000#/□"?

* Rivets with heads flattened to not less than $\frac{3}{8}$" show about 0.9 the strength of rivets with full heads, in tests. Rivets which have the heads flattened to less than $\frac{3}{8}$" should be considered to be the same as countersunk rivets. Some designers allow a tensile strength for rivets of not greater than one-half the single shearing strength. Obviously bolts are cheaper for such cases.

† The exact procedure is that used in the design of pins (see Index).

Prob. 27b. What is the double shear value of a $\frac{3}{4}''$ bolt at a unit stress of 8000#/□″? What is its bearing value on a $\frac{5}{16}''$ plate?

Prob. 27c. What is the tensile strength of a $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ angle with two $\frac{3}{4}''$ rivet holes out?

28. Standard Beam Connections.

When a beam frames into another or into a girder, a pair of angles is the usual form of the connection. Occasionally a pair of angles is used to frame a beam or girder into a column but this practice is not common. Structural steel fabricating companies usually standardize such connections as far as possible, as the cost of fabrication is reduced by making them in quantities irrespective of when they are to be used. The saving gained by the omission of special fabrication, the time saved, and the making of the details more than offset the extra weight and rivets that a standard connection may have in excess of that for a connection actually required for any given case. The standards are composed of certain sized angles of a given length to accommodate a definite number of rivets, and they are grouped according to the various beam sizes. These standards vary with different structural companies, but they are all made with the same purpose in view. Figure 54 shows a set of typical standards and Table 25 gives their safe capacities under varying conditions which agree well with tests. All the holes are punched $\frac{13}{16}''$ (for $\frac{3}{4}''$ rivets and bolts), and the lengths of the angles are planned to at least clear the fillets of the largest size of beam in the group. The widths of the legs of the angles are made large enough to accommodate the rivets with sufficient edge distances.

The safe capacity of any beam connection depends upon the value of the web connection, and the value of the outstanding legs of the connection angles. The first is controlled by the enclosed bearing of the rivets on the beam web (Art. 27) or by the double shearing resistance of the rivets. The unenclosed bearing of the rivets on the thickness of metal determined by the two angles practically never controls, as the sum of the thickness of the two angles is enough in excess of that of the web to offset the difference in the respective allowable stresses, namely 24,000#/□″ and 30,000#/□″. The thickness of the angles is purposely selected to accomplish this result. Figure 55 shows a test failure which bears out these relations. In determining the value of the outstanding legs of the connection angles, the detail designer must know whether the field connections are to be rivets or bolts (Art. 18). In either case, the strength is determined by single shear or by unenclosed bearing on the leg of the angle. In many cases, beams may not frame opposite, so that double shear should not

TABLE 25

LIMITING VALUES OF BEAM CONNECTIONS*

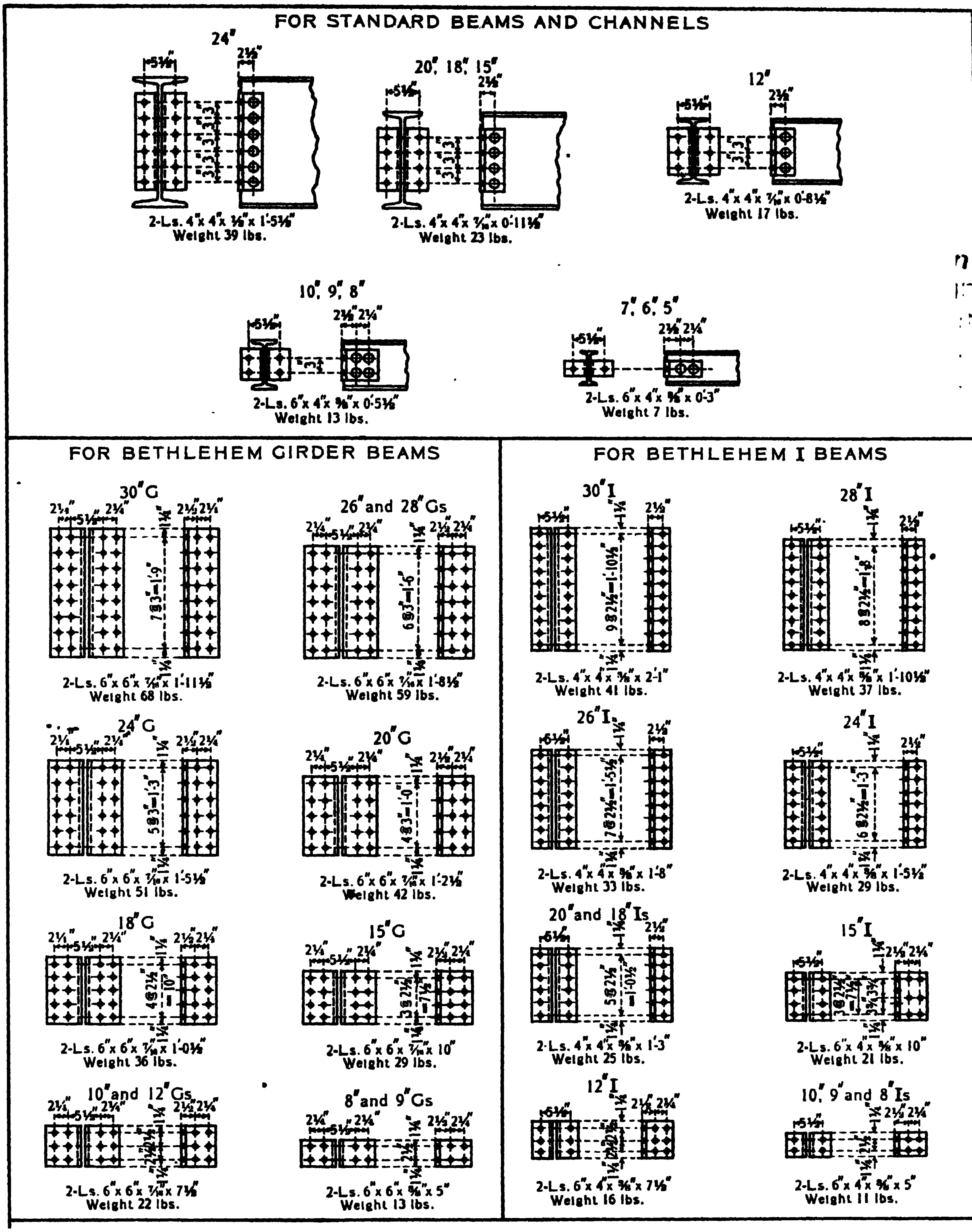
Beam		Value of Web Connection	Value of Outstanding Legs of Connection Angles	
Depth	Weight	Shop Rivets	Field Rivets	Field Bolts
Standard Beams and Channels†				
24	79.9	63,600	53,000	42,400
20	65.4	42,400	35,300	28,300
18	54.7	41,400	35,300	28,300
15	42.9	36,900	35,300	28,300
12	31.8	23,600	26,500	21,200
10	25.4	27,900	17,700	14,100
9	21.8	26,100	17,700	14,100
8	18.4	24,300	17,700	14,100
7	15.3	11,300	8,800	7,100
6	12.5	10,400	8,800	7,100
Bethlehem Girder Beams‡				
30	181	106,500	128,000	96,000
28	165	88,000	112,000	84,000
26	151	88,000	112,000	84,000
24	121	70,400	96,000	72,000
20	113	54,000	80,000	60,000
18	93	54,000	80,000	60,000
15	74	38,800	64,000	48,000
12	55.5	25,100	48,000	36,000
Bethlehem I Beams‡				
30	121	72,200	72,000	54,000
28	106	63,000	64,000	48,000
26	91	53,700	56,000	42,000
24	73.5	44,500	48,000	36,000
20	59.5	35,200	40,000	30,000
18	49.0	35,200	40,000	30,000
15	38.5	26,100	32,000	24,000
12	28.5	17,500	24,000	18,000
10	23.5	10,700	16,000	12,000

enter into the calculations of the field connection, which may be used for a beam framing in on one side only. Single shear usually controls, as the angles would have to be less than $\frac{3}{8}''$ thick to have the bearing on them control (Table 24). It will be noted in Fig. 54 that no angles less than $\frac{3}{8}''$ thick are used, so that the bearing does not control. No bending is figured on the web rivets, as the eccentricity is small and it can safely be neglected in the average case. The maximum allowable end reaction for a standard connection is governed by the lesser of the values determined by the web connection and the outstanding legs, — the latter being governed by whether field rivets or field bolts are

* No moment considered. This is given practical protection by the friction between the webs and angles. Values are controlled by minimum size beam of given depth. Connections may be used for heavier beams, using the above values.

† Same connection angles may be used for corresponding depths of channel. Based on Carnegie "Pocket Companion," Single-Shear Shop Rivets 12,000#/□″, Field Rivets 10,000#/□″, Field Bolts 8000#/□″, Enclosed Bearing, 30,000 #/□″.

‡ Based upon Eastern Bridge & Structural Steel Co.'s standards.



Weights do not include rivets for shop and field connections.
All holes 13/16" diam. for ¾" rivets or bolts.

FIG. 54

used. If the end reaction of a given beam exceeds the allowable, as discussed, a standard connection cannot be used, and a special connection will have to be designed (Art. 29). The standard angles are generally satisfactory for the majority of beams and only in special cases is it necessary to use a stronger connection.

a beam is tied in and the value of g indicates the distance from the top of the beam detailed to the top row of holes in its web. The difference in the amount of drafting required to detail the beam is obvious. The template shop can make the templates, as it also is familiar with such details. A value of $\frac{1}{4}''\pm$ is subtracted from the out to out dimension for each

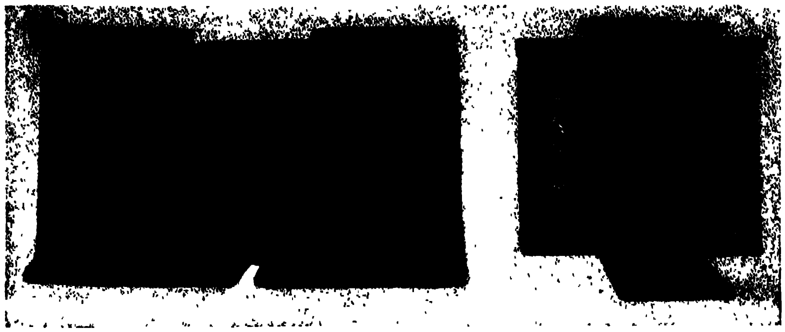


Fig. 55

Illustrative Prob. 28a.* Check the values given in Table 25 for a 24 I 79.9 standard beam connection. Thickness of web = 0.5″.

Web connection. The calculations are based upon the web thickness of the minimum beam $\frac{3}{4}''$ rivets.

Enclosed bearing = 30,000 × $\frac{3}{4}$ × $\frac{1}{2}$ = 11,240#
Double shear = 12,000 × 2 × 0.44 = 10,600#
Unenclosed bearing on 2 angles = 24,000 × 2 × $\frac{1}{2}$ × $\frac{3}{4}$ = 18,000# (This value is never calculated in an actual case).
10,600# controls. 6 Shop rivets shown.
6 × 10,600 = 63,600#, value of web connection.

Outstanding Legs. — Field rivets (or turned bolts).

Single shear = 10,000 × 0.4418 = 4420#
Unenclosed bearing on angle = 20,000 × $\frac{3}{4}$ × $\frac{1}{2}$ = 7500# (This value need not be calculated in an actual case).
4420# controls. 12 open holes shown.
12 × 4420 = 53,000#, value of o.s. legs.

Field bolts (rough bolts).

Single shear = 8000 × 0.4418 = 3530#
Unenclosed bearing on angle = 16,000 × $\frac{3}{4}$ × $\frac{1}{2}$ = 6000# (This value need not be calculated in an actual case).
3530# controls. 12 open holes shown.
12 × 3530 = 42,400#, value of o.s. legs.

Thus for a 24 I 79.9, when rivets are the field connection, the maximum end reaction the standard connection can sustain is 53,000#. When bolts are the field connection, the value is 42,400#. It should be noted by a study of Table 25 that the field connections do not always control the allowable end reaction.

Figure 56 shows a **typical beam detail** used when standard connection angles are employed. The standard angles for the 18″ beam are designated by 18 *K*. When a beam with standard connection angles frames into the beam being detailed, the indication given is all that is necessary. The center-line of such

* This problem may also be solved by reference to Table 24.

end connection to establish the cutting length of the beam itself. This value is adjusted so that the cutting length is to the nearest $\frac{1}{2}''$, bearing in mind that sufficient edge distance must be maintained on the rivet holes in the end of the beam (also see Pl. 4).

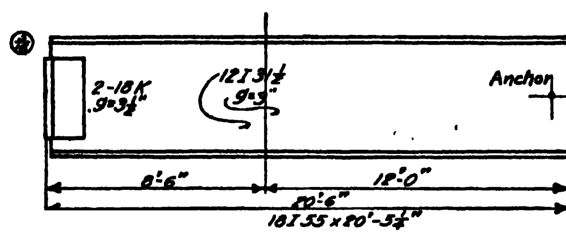


Fig. 56

Beams are usually framed **flush top** (F.T.), or **flush bottom** (F.B.). When this is done, **coping** is required, as illustrated in Fig. 57 (*a*) and (*b*). The coping is done by a special machine which cuts the flange to standard dimensions to clear any given beam. When a small beam frames into a larger one at an intermediate level, as shown in (*c*), no coping is obviously required. It is common practice to make the elevation of girders 2″ higher than that of floor beams (dimension "*a*" in the figure) to purposely avoid coping. Occasionally a beam

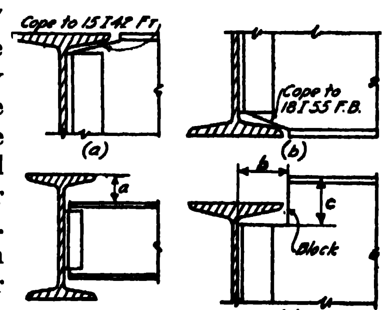


Fig. 57

frames into another which is shallower or at a different elevation. This requires **blocking** at the end of the beam, as illustrated in Fig. 57 (*d*), so that the connection angles may frame in. The dimensions "*b*" and "*c*" are determined by the necessary clearance.

Seat angles are sometimes used to support the ends of beams which frame into deep members, and usually when beams frame into columns (Fig. 30).* These are an advantage in erection because the beam may be swung and bolted into position more readily and riveted later, whereas a beam with standard connection angles has to be at least temporarily connected with bolts while it is held in position at the same time. At times the erection in this case is quite difficult to accomplish. Sometimes an **erection seat** is used, which is bolted ½″ below the beam's final position. When the connection angles are riveted into proper place, the seat may be removed. It is sometimes left in place if it does not interfere with the fire protection materials. No part of the reaction should ever be calculated as taken by a seat angle and the remainder by connection angles because they do not act simultaneously and it is impossible to estimate what proportion of the load would be resisted by each. Consequently seat angles should be avoided if possible when connection angles are used. This should not be confused with the use of an erection seat which is proper in any case, as no load is calculated to be taken by it, and it can receive no load if the beam is detailed and erected properly.

Prob. 28b. Check the strength of a standard connection for an 18 I 54.7 for field rivets and also for field bolts, as given in Table 25.

Prob. 28c. A 12 I 31.8 has an end reaction of 25,000#. Field connections to be rivets. Can a standard connection be used?

29. Unusually Heavy Beam Connections.

If a standard connection is not sufficient to carry the end reaction of a large or heavily loaded beam, a special connection must be provided, employing the same principles of design as used for standard connection angles. The number of shop rivets and field rivets or bolts will be larger, and the angles may have to be of a greater size to accommodate them. For all beam connections, ¾″ rivets are usually employed, although ⅞″ rivets may be used if a more economical design can be obtained. It is also possible to use standard connection angles for excessive reactions by changing the punching from $\frac{13}{16}$″ to $\frac{15}{16}$″ and using ⅞″ rivets. This may be noted as 18⅞ *K*, for a standard 18″ beam connection with ⅞″ rivets, and so on, in the details.

* Inasmuch as seat angles are riveted to the supporting member, they are detailed with it, and the design of such details is considered in connection with those members (see Index — seat angles).

Illustrative Prob. 29a. If a 24 I 79.9 had an end reaction of 61,000#, design the required end connection. Field connection to be riveted, — ¾″ rivets (Fig. 58).

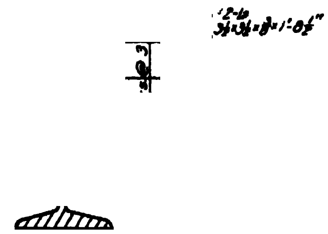


FIG. 58

The standard connection is good for only 53,000# (Table 25) and therefore cannot be used.

Web Rivets — controlling value = 10,600#

$$\frac{61,000}{10,600} = 5+ \text{ say } 6.$$

Field Rivets — controlling value = 4420#

$$\frac{61,000}{4420} = 13+ \text{ say } 14.$$

From structural tables, clear distance between fillets = 20¾″†
2 × edge distance for outside rivets = 2½
18¼″

7 Field Rivets needed each side = 6 spaces
18 ÷ 6 = 3″ available for each space.

Therefore 3″ spacing may still be used in the angles. 7 shop rivets are shown through the web to match the 7 holes in each outstanding leg. This makes a simpler detail, a simpler template, and the angles are interchangeable. If more rivets were needed to carry the end reaction, then a 2¾″, 2½″ or even 2¼″ spacing might be used. The number of spaces required can be divided into the available clear distance between fillets and the spacing determined as has been shown. If the rivets cannot be driven in one line, then a 6″ legged angle may have to be used, especially for the outstanding legs. The rivets through the web preferably should line up with those in the outstanding legs, even if one or two extra rivets are necessary to do so, for reasons previously given.

Illustrative Prob. 29b. A 12 I 31.8 has an end reaction of 40,000#. Field connections bolted. Design the end connection for ¾″ rivets. (Fig. 59.) The standard can carry only 21,200# and hence it cannot be used. Web t = 0.35″.

Web Rivets.

Enclosed bearing = 30,000 × ¾ × 0.35 = 7870#
Double shear = 10,600#
7870# controls

$$\frac{40,000}{7870} = 5+ \qquad \text{Use 6 rivets or 5 spaces.}$$

From structural tables, clear distance = 9¾″
2 edge distances = 2¼
7½

7½ ÷ 5 = 1½ Hence must use 2 lines of rivets.

† The distance from the top of the beam framed into, to the toe of the fillet may influence the limiting length of the angle if this beam is larger. If the length of angle just encroaches on the fillet, the corners of the angle may be ground off to clear the fillet. The angle probably might engage the fillet ⅛″ without much harm.

Field Bolts — controlling value = 3530#

$\frac{40,000}{3530} = 11+$ Use 12. Must use 2 lines of rivets.

Hence use 6 × 6 angles.

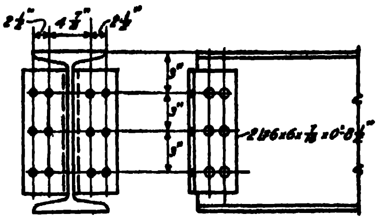


FIG. 59

When a double gauged angle is used, it should not be made too thin on account of the shear and bending that indirectly come upon it. The number of rivets should never be reduced from what is theoretically required for they receive some bending stress. The amount of this stress is rather difficult to estimate.

For shear each angle gets 20,000#

$\frac{20,000}{10,000} = 2.0\square''$ required. Net section $= [7\frac{1}{2} - (3 \times \frac{7}{8})]\,t$
$= 4.88\,t$

$\frac{2.0}{4.88} = t = 0.41''$. Hence use $\frac{7}{16}''$ angles.

Prob. 29c. Design a beam connection for a 15 I 42.9 if it has an end reaction of 43,000#. Field connections, rivets.

30. Special Framing and Skew Connections.

In special cases, beams are often framed into each other at angles other than 90°, or they are said to be "at a skew." In such a case the connection angles, either standard or special, are "**opened**" or "**closed,**" parallel to the axis of the beam "framing in," as illustrated in Fig. 60 (*a*). When *t*, as shown, is $\frac{5}{8}''$ or less, standard connection angles may be used, providing the reaction to be carried is within the allowable. For values of *t* from $\frac{11}{16}''$ to $1\frac{1}{8}''$, special angles must be used on account of the clearance necessary to drive the rivets. Note how the flange of the beam framing in is blocked. The detail in (*b*) requires sawing and should not be used, but (*c*) or (*d*) should be employed instead, as the beams may be blocked with comparatively small expense. When the figures determining the bevels, as *b* in Fig. 60 (*e*), are less than 3″, standard angles may be used. Otherwise bent plates must be employed.* For special angles, the bevels may be increased somewhat by re-spacing the holes. In any case, bevels greater than 45° produce very awkward framing and they should be avoided when possible. The design of such angles and plates is similar to that in the previous discussion.

* Valuable information in this connection may be found in M. S. Ketchum's "Structural Engineers' Handbook" — McGraw-Hill Book Publishing Co.

Where the clearance is small, in framing around certain openings, or in framing a double beam, one-sided connections may be required. The same principles of design may be used as for previous cases.

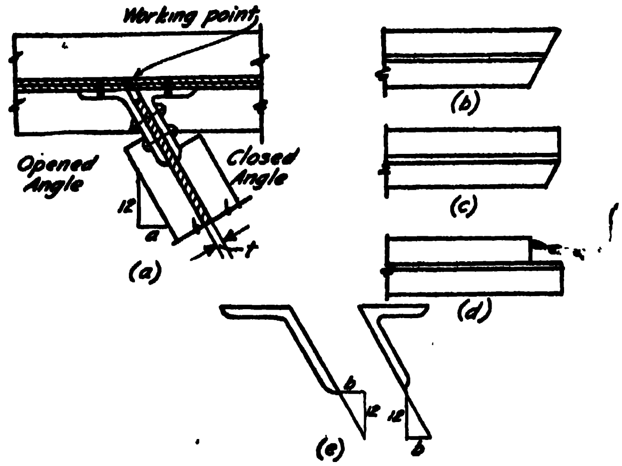


FIG. 60

Illustrative Prob. 30a. A 15 [33.9 has an end reaction of 30,000#. Field connections riveted. Design a one-sided connection for $\frac{3}{4}''$ rivets. (Fig. 61.)

Web Rivets

Unenclosed bearing $= 24,000 \times \frac{3}{4} \times 0.4 = 7200\#$
Single shear $= 12,000 \times 0.4418 = 5300\#$.

To develop single shear the angle must be

$$\frac{5300}{24,000 \times \frac{3}{4}} = t = 0.29'' \text{ thick.}$$

Hence try $\frac{3}{8}''$ angle.
5300# controls.

$$\frac{30,000}{5300} = 5+ \text{ say 6 rivets.}$$

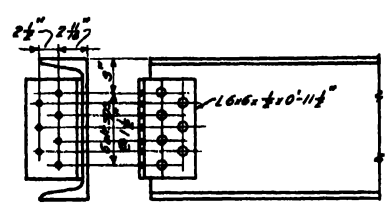


FIG. 61

Field Rivets

Unenclosed bearing $= 20,000 \times \frac{3}{4} \times \frac{3}{8} = 5630\#$
Single shear $= 10,000 \times 0.4418 = 4420\#$

4420# controls.

$$\frac{30,000}{4420} = 6+ \text{ say 7 or 8 rivets.}$$

The clear distance between fillets $= 12\frac{1}{4}''$
2 edge distances $= 2\frac{1}{2}$
$9\frac{3}{4}''$

$9\frac{3}{4} + 5$ spaces $= 1.95''$. Use 2 lines for web rivets.
Similarly use 2 lines for field rivets.

Seven rivets are shown through the web to keep the angle balanced. 8 field rivets could be used to keep symmetry if desired.

$$\frac{30,000}{10,000} = 3.0\square'' \text{ required for shear in angle}$$

$$\text{Net section} = (11\tfrac{1}{2} - 4 \times \tfrac{7}{8})\, t = 8\, t$$

$$\frac{3.0}{8} = t = \tfrac{3}{8}''. \quad \text{Requires } \tfrac{3}{8}'' \text{ angle.}$$

It probably would be better practice to make the angle ½" thick, because of some bending, and the fact that it is a one-sided connection.

A feature which has been receiving attention for the past few years is that of fused **welded joints**, as applied to structural work. Such work has been done for oil and water tanks to some extent. Insufficient data exist at present to warrant wholesale adoption for structural joints. However, progress is being made in this direction and a few smaller buildings have had the joints of the frame welded,* and additional experimentation and data may lead to more common usage in the future.

Riveted joints only develop about 75% of the strength of the member, while welded joints, if properly made, may be made to equal the full strength of the member with a 10% to 25% saving in weight and consequent reduction in cost. Two types of welded joints, having the same sectional area of weld, have been employed, namely, a *V* and a double *V*. The latter is claimed to be from 25% to 50% stronger as shown by tests.† Other tests show an ultimate shearing strength of about 36,000#/□",‡ and hence with a factor of safety of 4, a working stress of 9000#/□" corresponds.

Some of the advantages which would result if welded joints were used are: (1) simpler drawings, (2) elimination of parts of the template work, steel marking, and punching of holes, (3) less accurate alignment of parts is required, and (4) less possible movement of the parts and hence less bending strain.§ A disadvantage is that poor welds are difficult to detect.

Prob. 30b. Design a one-sided connection for a 12 [20.8. Field connections, rivets. End reaction = 12,000#.

Prob. 30c. Make a sketch of a 12 I 31.8 framing into an 18 I 54.7 at an angle of 60° with the axis of the girder. End reaction = 30,000#. Can standard connection angles be used if the field connections are rivets? Show proper bevels for opening and closing the angles, and the blocking details if the beams frame flush top.

* Refer to data compiled by the American Bureau of Welding.

† Tests in England in behalf of Lloyd's Register of Shipping, Bureau of Buildings, City of New York.

‡ Tests at Union College, Schenectady, N. Y.

§ For a further discussion, refer to "Buildings, Engineering and Contracting" for Oct. 28, 1925.

CHAPTER 4

SPECIAL COMBINATIONS OF ROLLED BEAMS

31. Multiple Beams.

Two or more beams may be used side by side to carry heavy loads, to provide a broad surface to carry walls, or to save headroom, and the like. Such beams may result in poor details where other beams frame into them because of the difficulty of providing the connections. However, the increased width helps to provide lateral stiffness. The beams may be held together by bolts and separators of some form, as in Fig. 62 (*b*), or by plates, as in (*a*). The latter is not a common detail and it cannot be used where there is moisture, because the whole section is not accessible for painting and inspection. (For a discussion of the design of this type of multiple beam, see Art. 86.) Figure 62, (*c*) and (*d*), shows other typical multiple beams.

The design of common multiple beams is made possible by the application of the general flexure formula. The total strength of two beams of the same size, as in type (*b*) for example, is the same as twice that of one beam. It is important in such a case to have each beam take its half of the load, and the arrangement of the beams with respect to the materials to be carried should make this possible in order to eliminate any torsion. If the beams must act as a unit, such as when carrying a concentrated load, the separators* should be of the diaphragm type. If the load on each beam is not the same, each beam should be designed separately to carry its corresponding load.† If possible, the beams should be kept of the same depth. In many cases, architectural features may not allow this, in which case the properties of a section such as shown in Fig. 62 (*c*) must be carefully investigated. The clear distance between the flanges should be not less than 3″ preferably, when beams are to be framed into the double beams, so that the connections may be made (see also diaphragm separators, Art. 32). If the distance is less than 3″, handholes, not less than 6″ in diameter, should be provided. These should be spaced so that a man does not have to reach more than 2′-0″ in any direction to insert a rivet or bolt. If the beams must be close together, 6″ × 6″ connection angles should be used on the outside faces of the beams.

Illustrative Prob. 31a. Select a pair of beams to carry a uniform load of 1200#/ft., caused by a 12″ brick wall, on a 12′-0″ span. 8″ Bearing at ends.

$M = 1.5\,w \cdot L^2 = 1.5 \times 1200 \times (12)^2 = 259{,}000''\#$

$\frac{I}{c} = \frac{259{,}000}{16{,}000} = 16.2''^3 \qquad \frac{16.2}{2} = 8.1''^3$

Use 2-7 *II* 15.3 × 12′-8″.

$b = 3.66''$
See Art. 32.
Space separators
6″, 3′-10″, 4′-0″, 3′-10″ *and* 6″ = 12′-8″

Use *beams* 4½″ *o.c.* (8⅛″ out to out of flanges with standard c. i. seps., or beams 6″ o. c. (9⅝″ out to out of flanges) with 1″ gas pipe and ¾″ rod seps.).

Illustrative Prob. 31b. Design two beams to carry 9′-0″ of 12″ brick wall over an 11′-0″ span when the beam toward the inside of the building must also carry 6′-0″ of floor carrying a total load of 200#/□′. 6″ Bearing.

It should be evident that in this case each beam must be designed separately.

Outside beam

Brick work 120#/c.f.

$$M = 1.5\,w \cdot L^2 = 1.5 \times \frac{9 \times 120}{2} \times (11)^2 = 98{,}000''.$$

$$\frac{98{,}000}{16{,}000} = 6.12''^3 \qquad \textit{Use } 6\ I\ 12.5.$$

Inside beam

$$\text{Load} = \frac{9 \times 120}{2} = 540 \text{ from wall}$$

$$6 \times 200 = \underline{1200} \text{ from floor}$$

$$1740\#/\text{ft.}$$

$$M = 1.5 \times 1740 \times (11)^2 = 313{,}000''\#$$

$$\frac{I}{c} = \frac{313{,}000}{16{,}000} = 19.51''^3 \qquad \textit{Use } 10\ I\ 25.3.$$

Space separators 6″, 5′-3″, 5′-3″ *and* 6″ = 11′-6″.
Use separators similar to Fig. 67 (*b*).

Prob. 31c. Select a pair of beams to carry a load of 12,000# concentrated at the middle of a 16′-0″ span. What recommendations should be made for connecting the beam bringing in the 12,000#? What clear spacing between the flanges should be used? Are handholes necessary?

Prob. 31d. Design two beams to support 8′-0″ of 12″ brick wall (120#/c.f.) on a 12′-0″ span when one beam must also carry 7′-0″ of floor carrying a total load of 250#/□′.

* Since separators vary according to the conditions surrounding any problem they will be discussed as a separate entity (Art. 32), although they are a part of multiple beam design as well as of other members.

† Care should be taken, especially for double beams used in interiors, to keep the deflections of each beam as nearly alike as possible, to prevent cracks in the finish. Diaphragms help to overcome unequal deflections due to unbalanced live loads.

32. Beam Separators.

Separators are used to hold the compression flanges of two or more beams in position, and to aid in making the beams act together. They should fit the flanges tightly so that they will help the beams to act together both vertically and horizontally. The usual limit established for the spacing, in members subjected to compression without induced sidewise bending, is 10 times the least dimension of the cross-section. Otherwise, the working stresses must be reduced (Art. 11). The flange widths, b, of the smaller I beams vary from 4″ to 6″. If d_s is the distance between separators, then

$$d_s = 10\,b, \quad \text{and} \quad 10 \times 4 = 40'' = 3'\text{-}4'' \quad \text{or}$$
$$10 \times 6 = 60'' = 5'\text{-}0''.$$

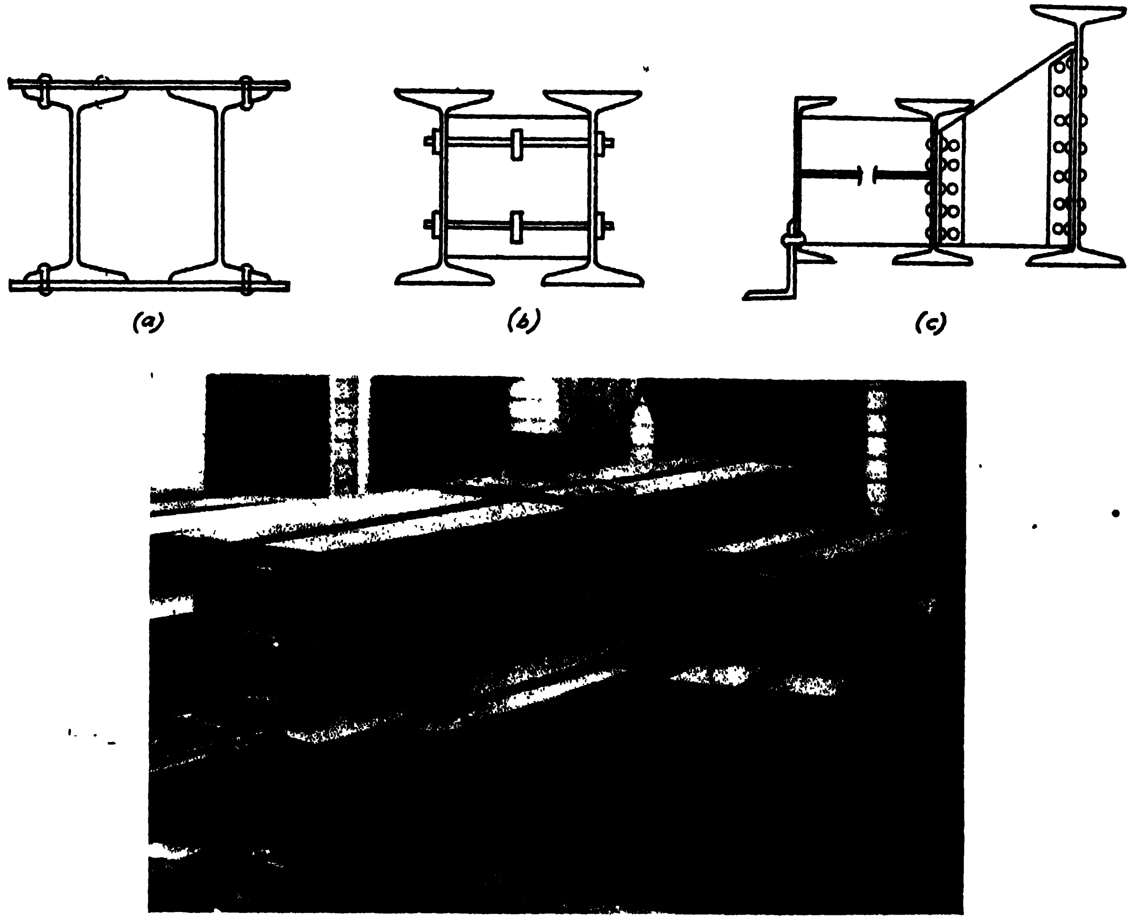
Accordingly, a rule often used in practice is:

SPECIFICATION CLAUSE

Spacing Separators shall be spaced from 4′-0″ to 5′-0″ on centers, depending upon the size and importance of the member, with 5′-0″ as a maximum

Separators should also be located about 6″ from the ends of the beams, and under any concentrated loads. **Beams less than 12″ deep** should have **one tie-bolt per separator,** while all **other** standard **beams,** and Bethlehem beams from 12″ to 24″ inclusive, should have **two tie-bolts,** and the **26″, 28″ and 30″** Bethlehem beams and girders should have **three tie-bolts** per separator.† There are several kinds of such fastenings used, each of which has merits for a particular use.

Cast-iron separators should be used principally as spacers, and not when the beams must act in unison, or where there are concentrated loads. They lend themselves readily to usual conditions, but they should not be used if lateral thrust or varying loads occur, because moments may be developed in the separators. Since cast iron is weak in tension, separators of this material should not be subjected to bending stresses. In all cases the edges



(a) (b) (c)

(d)*

FIG. 62

* Courtesy of New England Structural Co.

† Gas-pipe separators are used for 5″, 4″ and 3″ beams.

should be true and square. Figure 63 shows two standard proportions for cast-iron separators. The following relations are basic:

h is dependent upon the clear distance between the fillets,

d is dependent upon placing the bolts reasonably near the top and bottom of the beam, and

w is dependent upon a reasonable distance between the toes of the flanges and upon a reasonable distance center to center of beams.

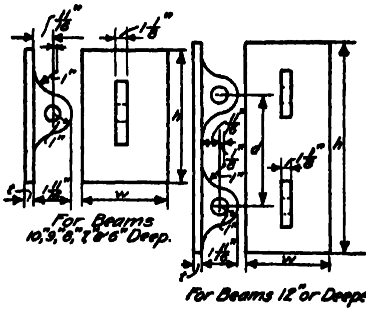


FIG. 63*

Separators of 1″ gas pipe and ¾″ rods are used as an alternate by some fabricators for cases similar to those in which cast-iron separators may be used. Figure 64 gives a standard arrangement. The following relations are true:

l_G = (c.c. Beams) − t, to the nearest $\frac{1}{16}$″, and

l_R = (c.c. Beams) + t + [thickness of 2 nuts (each = diameter of rod)] + (an adjustment allowance of 1″±), to the nearest ½″.

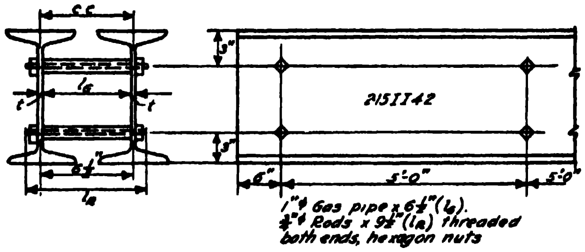


FIG. 64†

An advantage of this type of separators is the opportunity to vary the distance center to center of beams (c.c.). If the beams are to carry a wall, the distance out to out of flanges should be reasonably near but slightly less than the wall thickness (1″ in from each wall face is a good guide).

When concentrated loads occur, or when special rigidity is desired, some form of **diaphragm separator** should be used. Figure 65 gives an excellent detail used by some structural companies where the center to center distance between the beams does not have to be an accurate set dimension, such as for beams which are to carry walls and the like. Odd pieces of channel may be used as shown, or

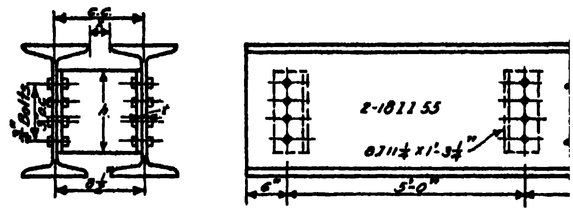


FIG. 65

pieces of I beam may be employed in a similar manner. The channel sections provide sufficient stiffness however, and require fewer bolts. The following relations are true for this type:

h = the clear distance between the fillets, and

(c.c. of Beams) = (depth of ⊏) + t.

The distance, x, must be sufficient to allow for the turning of the nuts on the bolts. This should be 2½″, minimum, and preferably 3″. If the two beams must be close together, the holes may be reamed and turned bolts used. If x may be made 3″ or greater, rivets can be used, although bolts are common. Figure 66 shows a detail of a built-up diaphragm separator.

If the beams to be held together are of different depths, **special separators** must be used. Figure 67 shows instances of this kind.

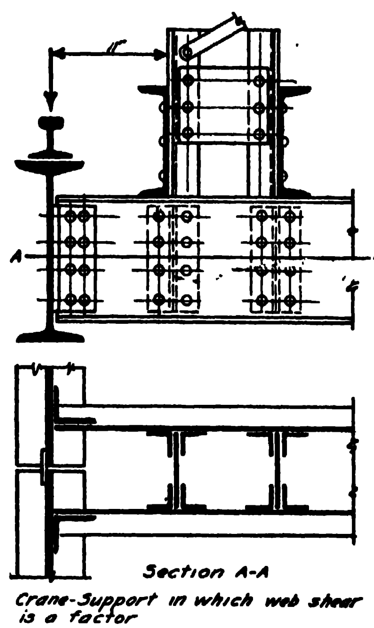
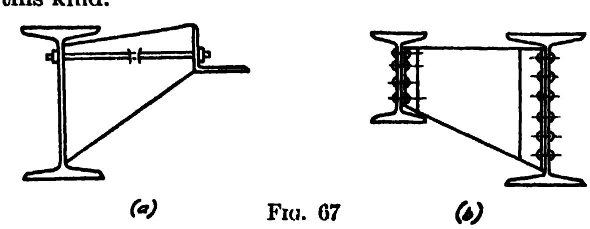


FIG. 66



(a) FIG. 67 (b)

Prob. 32a. What spacing should be used for the separators in a pair of beams 15′-0″ long? If the beams are 15″ deep, how many tie-bolts per separator should be used? If the beams are 15 I 42.9, refer to the Carnegie Pocket Com-

* Based upon the "Pocket Companion," Carnegie Steel Co. This book also gives the actual dimensions corresponding to the beam sizes. With a standard established for the separator, a standard distance center to center of beams results. These distances are also given in the handbook.

† Eastern Bridge & Structural Co., Worcester, Mass.

panion and make a sketch of the cast-iron separators to be used, giving the necessary dimensions. What spacing of beams would result?

Prob. 32b. As an alternate for Prob. 32*a*, determine the values of (c.c. beams) l_G, and l_R, for pipe separators, if the beams are to carry a 16″ wall. What type of separator should be used if the beams carry a concentrated load or unequal loads? What size of channels could be used instead of the gas pipe? What would be the length? What is the corresponding spacing of beams in this case? Make a sketch of a plate and angle separator for this case. What spacing of bolts should be used?

33. Compound Beams.

If extra strength is required and web plate material is not available, I beams may be compounded by placing one on top of the other, as indicated in Fig. 68 (*a*), if available headroom allows. Additional strength may be provided by adding cover plates top and bottom as in (*b*), or channels as in (*c*). The important features in such design, apart from that of providing for flexure, are:

(1) The horizontal shear at the plane where one beam rests upon the other must be taken care of by ample riveting,
(2) Precaution should be taken to have the webs strong enough in shear and buckling, and
(3) Lack of lateral support may control.

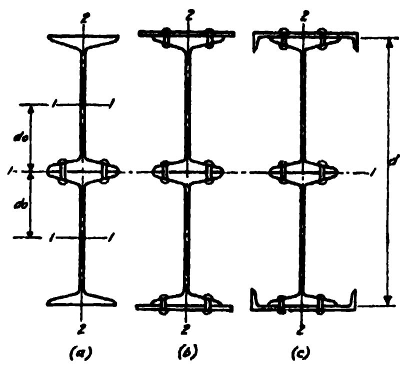


FIG. 68

Such beams are not commonly used as they are not generally as economical of material as plate girders, but they may be cheaper than the latter when fabrication costs are considered.

The lengths required for the cover plates in (*b*) or the channels in (*c*) may be found by the methods used for plate girders.* The section modulus of two beams, one over the other, is about 30% greater than that of the same two beams placed side by side. The design involves a "cut and try" method. A trial section may be obtained by calculating the section modulus required, dividing this by 1.3, and then selecting two beams to provide the latter value. The combined moment of inertia† of one beam over the other may then be calculated and the resulting stress compared with the allowable. The pitch of the rivets required may be calculated by the use of the general shear formula,

$$b \cdot q = \frac{V \cdot Q}{I} = \text{shear per linear inch (Art. 12)},$$

and the controlling value of the rivet, R, or

$$d_s \text{ (the rivet spacing in inches)} = \frac{R}{b \cdot q}. \qquad (S\text{–}16).$$

The minimum spacing of rivets is required at the ends of the span where the shear is maximum. Another way of calculating the pitch is to provide a sufficient number of rivets in a length equal to the overall depth of the compound beam, at the end, to develop the maximum shear, V. For practical purposes, the spacing at the ends of the beam should be not greater than 3″ o.c. for a distance equal to the depth of the compound beam, and the remainder of the rivets should not be spaced more than 6″ o.c. staggered.

Illustrative Prob. 33a. Design a compound beam to carry a load of 1000#/ft. on a 30′-0″ span. Assume beam to be laterally supported and 12″ bearing at the supports.

$$M = 1.5\, w \cdot L^2 = 1.5 \times 1000 \times (30)^2 = 1{,}350{,}000''\#$$

$$\frac{I}{c} = \frac{1{,}350{,}000}{16{,}000} = 84.2''^3 \text{ (Assume 18 I 54.9 is not available).}$$

$$84.2 \div 1.3 = 64.8''^3 \qquad \frac{64.8}{2} = 32.4''^3 \qquad \text{Try 2-12 I 31.8.}$$

Combined $I_0 = I + A \cdot d_0^2$

$$\begin{aligned} I_{1\text{-}1} \text{ for a 12 I 31.8} &= 215.8''^4 \\ A \cdot d^2 = 9.26 \times (6.0)^2 &= \underline{333.1} \\ & 548.9 \\ & \underline{\cdot\ 2} \\ & 1097.8''^4 \text{ gross} \end{aligned}$$

$$\frac{I}{c} = \frac{1097.8}{12} = 91.5''^3 > 84.2 \quad O.K.$$

$$V = 1000 \times \frac{30}{2} = 15{,}000\#$$

$$Q = A \cdot d_0 = 9.26 \times 6 = 55.56''^3$$

$$b \cdot q = \frac{15{,}000 \times 55.56}{1097.8} = 758\#/\text{inch.}$$

Controlling value for ¾″ rivets (Single shear) = 4420#

$$d_s = \frac{R}{b \cdot q} = \frac{4420}{758} = 5.84'' \text{ o.c. staggered.}$$

Use 3″ o.c. staggered for 2′-0″ each end and then 6″ o.c. staggered.

Shear. Average $= \dfrac{15{,}000}{24 \times 0.35} = 1780\#/\square''$

Hence *maximum shear O.K.*

Buckling.

$$f_b = 16{,}000 - \frac{120 \times 24}{0.35} = 7770\#/\square''.$$

$$R = 7770 \times 0.35\left(12 + \frac{24}{4}\right) = 48{,}000\# \text{ allowable.}$$

15,000# actual *O.K.*

Prob. 33b. Design a compound beam to carry a load of 1700#/ft. on a 38′-0″ span together with a load of 9000# concentrated at mid-span. Assume beam is laterally supported. It frames into columns with standard seat angles.

34. Riveted Beam Girders. (See Art. 86.)

35. Plate Girders.

The usual method, when sufficient section modulus can not be supplied by rolled shapes, is to employ built-up beams of plates and angles, called plate girders, which are discussed in Chap. 5.

* Arts. 51 and 52.
† The rivet holes occur at the "neutral axis" so that the gross section may be used, unless plates or channels are added to the flanges, or other holes occur at the point of maximum moment.

PLATE 8 PLATE GIRDERS READY FOR SHIPMENT

Courtesy of the New England Structural Company

PLATE 9 PLATE GIRDER IN PLACE
NEW JEWETT REPERTORY THEATRE BALCONY
BOSTON, MASSACHUSETTS

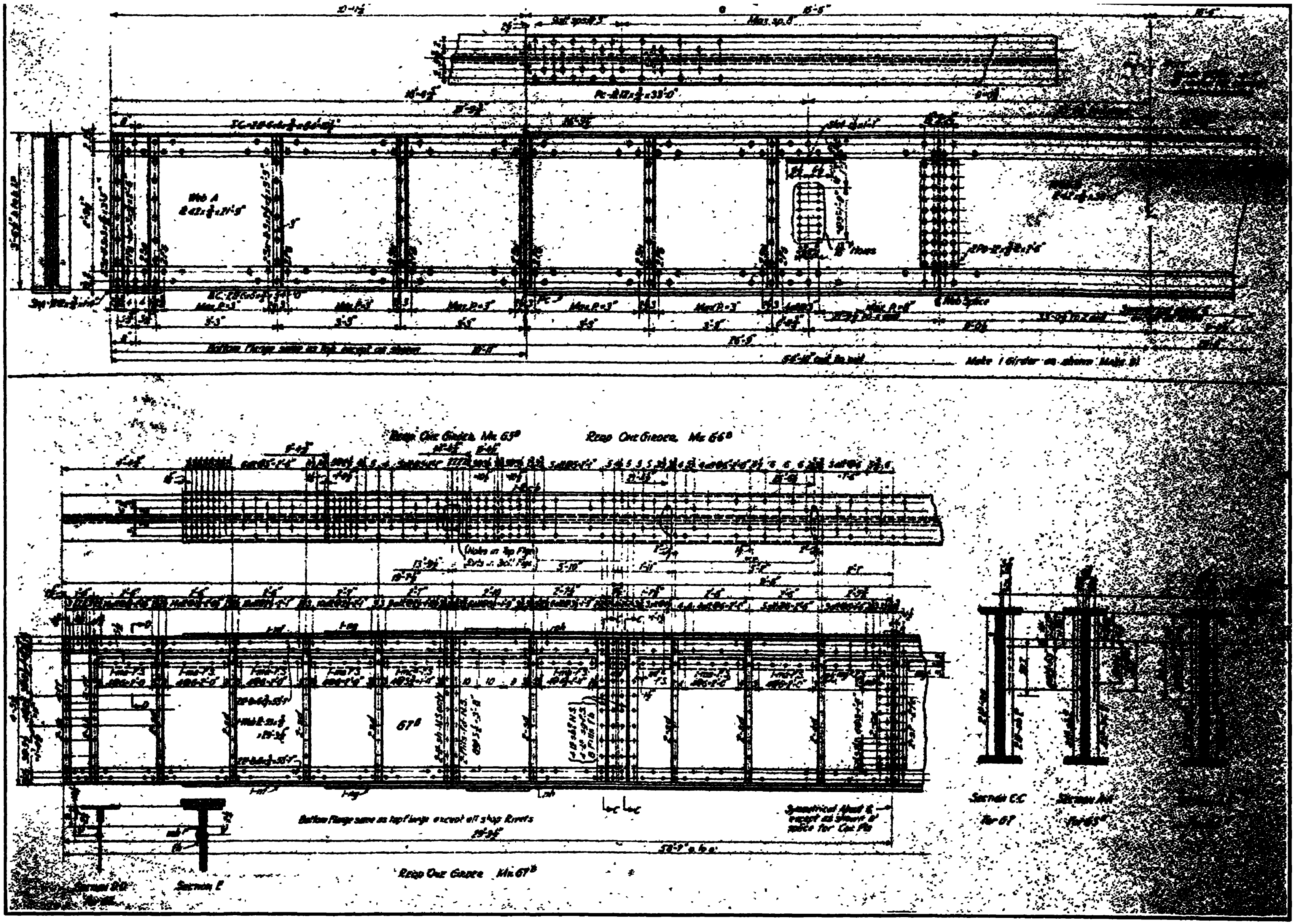


PLATE 10 TYPICAL PLATE GIRDER DETAILS

CHAPTER 5

PLATE GIRDERS

SECTION 5A

GENERAL CONSIDERATIONS

36. Rolled Sections versus Built-up Sections.

When rolled steel beams lack sufficient strength to carry the given load, or if the span is too great, other means to support the load must be provided, and a plate girder may be used. A plate girder is virtually a " built-up " I beam, made of structural plates and angles, or occasionally, channels. The provision of such a member involves more complicated computations than for the ordinary steel beam, as the component parts must be held together securely by rivets so that these parts will act in unison and without overstressing any of the joints. Plate girders are designed to resist bending, shear and the other internal stresses, as is the case for rolled beams, but because of their composite nature, they are more easily adapted to special cases. One can vary their depth and the sizes and arrangement of the parts and herein lies one of their valuable assets.

37. Usual Types.

Figure 69 shows several types of cross-sections for plate girders, showing some of the schemes for arranging the parts, which are named. A study of these names will reveal to some extent the functioning of the parts.

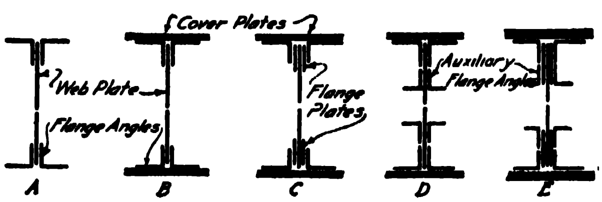


FIG. 69

Type *A* shows the simplest form in which the four angles correspond with the flanges, and the vertical plate with the web, of a rolled beam. Since metal becomes more efficient in resisting bending when it is placed farther away from the neutral axis (the basic idea of the I beam shape), unequal-legged angles are usually employed with the long legs horizontal. In this way, for the given area in the angle, the larger portion of the steel is at its maximum distance from the neutral axis. The smaller sized angles* (under 4×3 or 5×3) are not generally used, as their resistance at ordinary depths would not exceed the strength of the largest rolled beams, which are obviously cheaper for ordinary loads. If the flange area required should be larger than a selected pair of angles provides, cover plates may be added, as in the Type *B*. This method places additional steel where it will give the most bending resistance for the area added. It is not economical to resort to the maximum sized flange angles (8×8) before using cover plates, as the latter can be cut off at points along the girder where the moment does not necessitate their use, while the flange angles have to run to the ends. It is not, however, practicable to use cover plates when the angles are smaller than 5″ or 6″ as the cost of riveting of the plates is large in proportion to their weight. On the other hand, it is generally inadvisable to use too many cover plates, three or four being the limit.

SPECIFICATION CLAUSE

Center of Gravity Distance — The distance between the centers of gravity of the flanges shall not be greater than the distance back to back of flange angles.

Obviously too many cover plates will throw the center of gravity of the flanges outside of the plane of the backs of the flange angles. A good guide, although not inviolate, is that one-half the flange area should be in the flange angles unless the maximum sized angles must be used. This rule, if followed, also limits the number of cover plates. The length of the rivets passing through the flange angles and cover plates should not exceed four times the rivet diameter, if avoidable.

SPECIFICATION CLAUSES

Grip — The grip of the rivet shall not exceed four diameters of the rivet, nor shall the maximum thickness of metal in the flange exceed $4\frac{1}{2}''$.

(Alternate) — The maximum grip of the rivet shall be eight

* During the World War it was sometimes difficult to obtain beams in some localities and beams were " built up " with the smaller angles and plates from available stock. This was for an emergency, however, and only in special cases would details of this nature be used.

diameters of the rivet unless bending is figured on the rivet. (Similar to the design of pins, see Index.)

If a still greater flange area is needed, vertical flange plates may be introduced as shown in Type *C*. These plates should be as shallow as possible to give the required area, for, as before, the farther away the center of gravity of the plate is from the neutral axis of the girder, the more efficient it is in resisting bending. Another method used for the same purpose is to add extra angles, as shown in Type *D*. This is not as desirable, as it is less compact and the area in the inside angles is not at the maximum possible distance. These two ways may be used in combination, as shown in Type *E*, although this scheme is rather awkward and is used only where very large flange areas are required. The latter three combinations, Types *C*, *D* and *E*, are used for the purpose of making the distance between the centers of gravity of the flanges equal to or less than the distance " back to back of angles."

38. Special Types.

When girders are laterally unsupported, additional area may have to be added to the compression flange. This may be done as shown in Types *F* and *G* (Fig. 70). When girders carry very heavy loads, which develop excessive shears, or when large bearing areas are necessitated by walls, or when

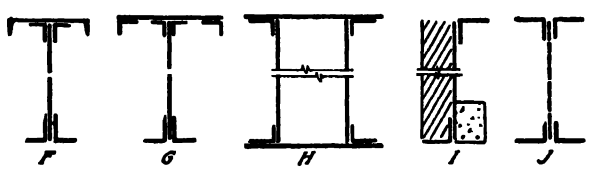


Fig. 70

headroom controls, box girders are sometimes used with two and possibly three web plates, as in Type *H* (Art. 84). For special conditions, such as the one shown, a type similar to *I* may be used. Type *J* is given to represent conventionally a girder in which the web plate is replaced by a system of lacing; such are often called latticed girders (Art. 85).

The prime essential in the selection of a type is to keep the section as simple as possible, and ornamental details should be avoided. The question of the location of the girder in the structure is also important, as the encasement by the materials which surround the steel will often determine maximum overall depths and breadths. The number of parts to be riveted should be kept to a minimum, for often excess weight in heavier angles is offset by additional fabrication costs when light angles and cover plates are used.

39. Typical Design Example.

It is desired to provide a steel girder for the following conditions:

Span, center to center of bearings, 50′-0″.
Girders 14′-0″ on centers, and they are laterally supported.
Uniform Load. End supports of concrete.

L.L. =	200	14 × 327 =	4600#/ft.
Fin. Flr. =	3	Bm &	
Sub Flr. =	3	Haunch =	300
2″ Cinder			4900#/ft.
Fill =	16		
8″ Slab =	100		
Plastered			
Ceiling =	5		
T.L. =	327#/□′.		

$$M = 1.5\,w \cdot L^2 = 1.5 \times 4900 \times (50)^2 = 18{,}340{,}000''\#$$

$$\frac{I}{c} = \frac{18{,}340{,}000}{16{,}000} = 1146''^3.$$

No standard or Bethlehem rolled section is large enough. Hence a plate girder will be used.

$$w = 3 + \frac{L}{4.25} = 3 + \frac{50}{4.25} = 14.8\#/\square'$$

14 × 327 = 4600#/ft.
Gdr. = 14.8 × 14 = 210
T.C. F.P. = 200 (Fireproofing)
5010#/ft., say w = 5000#/ft. (total).

$$V_{max.} = \frac{w \cdot L}{2} = \frac{5000 \times 50}{2} = 125{,}000\#$$

$$A_{WN} = \frac{125{,}000}{12{,}000} = 10.4\square''$$

$$A_W = \frac{4}{3} A_{WN} = \frac{4}{3} \times 10.4 = 13.9\square''$$

Depth of girder $\frac{50}{10}$ = 5′-0″ O.K. for headroom.

60 + ½ = 60½″ b · b · ∟s Use 60″ web plate.
60½ − 5 = 55½″ unsupported distance between flange angles.

$\frac{55\frac{1}{2}}{160}$ = 0.346 O.K. for Specifications.

Use 60 × ⅜ web plate.

$\frac{13.9}{60}$ = 0.232″ O.K. for shear.

$$M = 1.5\,w \cdot L^2 = 1.5 \times 5000 \times (50)^2 = 18{,}760{,}000''\#$$

Assume d_e = 60″

$$\text{Flange stress} = \frac{18{,}760{,}000}{60} = 312{,}500\# = F$$

$$\text{Net area of flange, } A_{FN} = \frac{312{,}500}{16{,}000} = 19.53\square''$$

⅛ A_W available for flange material.
A_W = 60 × ⅜ = 22.5□″ ⅛ A_w = ⅛ × 22.5 = 2.81□″
19.53 − 2.81 = 16.72□″ still to be supplied.
One-half should preferably be in flange angles

$$\frac{16.72}{2} = 8.36\square''$$

Try 2-6 × 4 × ½ ∟s. Use ⅞″ rivets.
Net area (2 holes out of each angle) = 7.5□″
16.72 − 7.5 = 9.22□″ to be supplied by cover plates.
Try 14″ C.Pl.

14 × ½ = 7.00□″	9.22
(2 holes out)	6.00
2 × 1 × ½ = 1.00	3.22□″
6.00□″	

$14 \times \frac{3}{8} = 5.25$
$2 \times 1 \times \frac{3}{8} = \underline{0.75}$
4.50

$\frac{1}{8}$ of $60 \times \frac{3}{8}$ web Pl. $= 2.81$
2 L^s $6 \times 4 \times \frac{1}{2}$
(2 holes out each L) $= 7.50$
$1\text{-}14 \times \frac{1}{2}$ C.Pl.
(2 holes out) $= 6.00$
$1\text{-}14 \times \frac{3}{8}$ C.Pl.
(2 holes out) $= \underline{4.50}$
Actual Total Area $= 20.81\square''$ } O.K.
Required Area $= 19.53$

Check center of gravity location (Fig. 71 (*a*)).

$$4.5 \times 30.94 + 6 \times 30.5 + 7.5 \times 29.26 - 18\,x = M_{A\text{-}A} = 0$$

$$x = 30.11''$$

$2 \times 30.11 = 60.22'' = d_e$
b. b. of L^s $= 60.5''$ } O.K. for Spec.

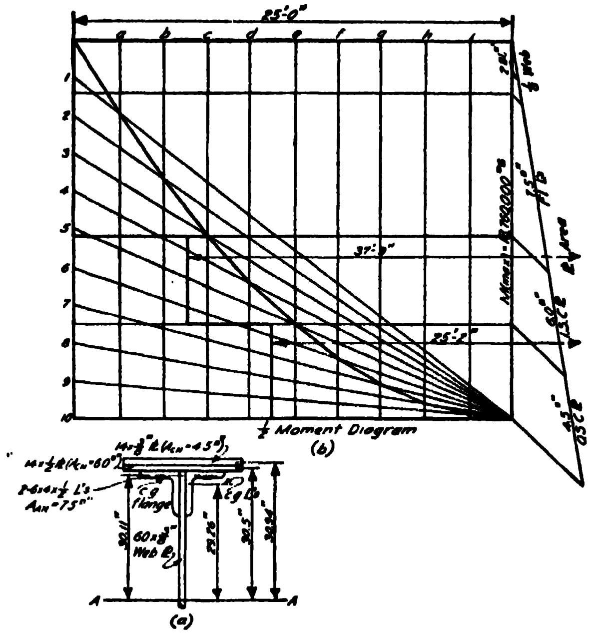


Fig. 71

Cover Plate Lengths (Fig. 71 (*b*) — Graphical Solution).

Analytical Check. A (total) at center $= 20.81\square''$, A_{CN} of plates $= 10.5\square''$ and $4.5\square''$ respectively.

$$\text{Inside plate } x_1 = \sqrt{\frac{A_{CN} \cdot L^2}{A}} = \sqrt{\frac{10.5 \times (50)^2}{20.81}} = 35.6$$

$\underline{2.0}$
37.6
say 37'-6".

$$\text{Outside plate } x_2 = \sqrt{\frac{4.5 \times (50)^2}{20.81}} = 23.2$$

$\underline{2.0}$
25.2 say 25'-6"

Use $1\text{-}14 \times \frac{1}{2}$ C.Pl. $\times$ 37'-6"
$1\text{-}14 \times \frac{3}{8}$ C.Pl. $\times$ 25'-6".

Figure 72 shows a structural engineer's sketch.

DETAIL DESIGN

Stiffeners at End Bearings. $R = 125{,}000\#$

Use 5" o.s. legs. Net bearing length $= 5 - \frac{1}{2} = 4\frac{1}{2}''$

$$\frac{125{,}000}{20{,}000} = 6.25\square'' \qquad \frac{6.25}{4.5} = 1.39'' \text{ combined thickness}$$

$$\frac{1.39}{4} = 0.35''$$

Use $4\text{-}5 \times 3\frac{1}{2} \times \frac{3}{8}$ L^s.

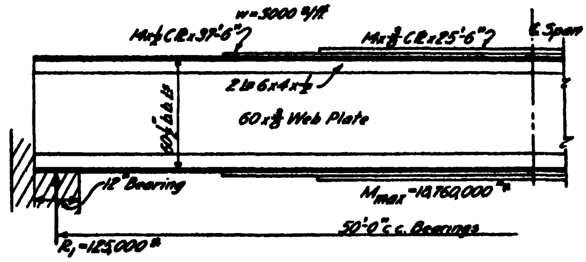


Fig. 72

Intermediate Stiffeners.

Unsupported distance between flange angles $= 60\frac{1}{2} - 5 = 55\frac{1}{2}''$

$$\frac{55\frac{1}{2}}{60} = 0.92'' \text{ required. Actual web thickness} = \tfrac{3}{8}''.$$

Intermediate stiffeners are required.

$$\text{Width of o.s. legs} = \frac{d}{30} + 2'' = \frac{60}{30} + 2 = 4''$$

Use $4 \times 3 \times \frac{3}{8}$ L^s.

V @ 3.0' $= 125{,}000 - 5000 \times 3.0 = 110{,}000\#$

$A_{WN} = 60 \times \frac{3}{8} \times \frac{3}{4} = 16.9\square''$

$$\frac{V}{A_{WN}} = \frac{110{,}000}{16.9} = 6{,}500\#/\square''$$

$$d_s = \frac{t}{40}\left(12{,}000 - \frac{V}{A_{WN}}\right) = \frac{0.375}{40}(12{,}000 - 6500) = 51.5''$$

Space 1st intermediate stiffener out 4'-0".

V @ 7.5' $= 125{,}000 - 5000 \times 7.5 = 87{,}500$

$$d_s = \frac{0.375}{40}\left(12{,}000 - \frac{87{,}500}{16.9}\right) = 64''$$

Space remainder at 5'-0".

Flange Rivets.

C.L. 1st panel 3'-0" from R_1 approximately
$V = 125{,}000 - 5000 \times 3 = 110{,}000\#$

No cover plate here. Hence $d_e = 2 \times 29.26 = 58.52''$ (Fig. 71 (*a*)).

$$V' = V \times \frac{A_{FN}}{A_{FN} + \frac{1}{8}A_W} = 110{,}000 \times \frac{7.5}{7.5 + 2.81} = 80{,}200\#.$$

$$\text{Horizontal shear per linear inch} = \frac{V'}{d_e} = \frac{80{,}200}{58.52} = 1370\#/\text{in.}$$

$$\text{Local load per linear inch} = \frac{5000}{12} = 417\#/\text{in.}$$

Resultant shear $= \sqrt{(1370)^2 + (417)^2} = 1438\#/\text{in.}$
Double shear per rivet $= 14{,}440\#$
Bearing $\frac{7}{8}''$ rivet on $\frac{3}{8}''$ plate $= 7880\#$

$$\frac{7880}{1438} = 5.48''.$$

Use 5" pitch — first panel.

C.L. 2nd panel 7'-6" from R_1 approximately
$V = 125{,}000 - 5{,}000 \times 7.5 = 87{,}500\#$. 1 C.Pl. here.
$d_e = 2 \times 29.7 = 59.4''$.

$87{,}500 \times \frac{13.5}{16.31} = 72{,}300\#$. $\frac{72{,}300}{59.4} = 1220\#/\text{in.}$

Neglect local vertical load.

$\frac{7880}{1220} = 6.46''$. Use 6" pitch all other panels.

Figure 73 shows a more complete engineer's drawing such as some engineers use.

$$M_W = \frac{s \cdot t \cdot d^2}{6} = 2{,}530{,}000 = \frac{13{,}860 \times t \times (52)^2}{6}$$

$t = 0.403''$ for 2 plates for moment

$A_{WN} = 16.9\square''$, $52 \times t \times \frac{3}{4} = 16.9$,

$t = 0.43''$ for 2 plates for shear

Use $\frac{7}{16}''$ min. thickness.
Use 2-52 × $\frac{7}{16}$ splice plates.

$\frac{202{,}000}{7880} = 25+$ rivets for shear. Assume vertical spacing shown in Fig. 74 (*b*).

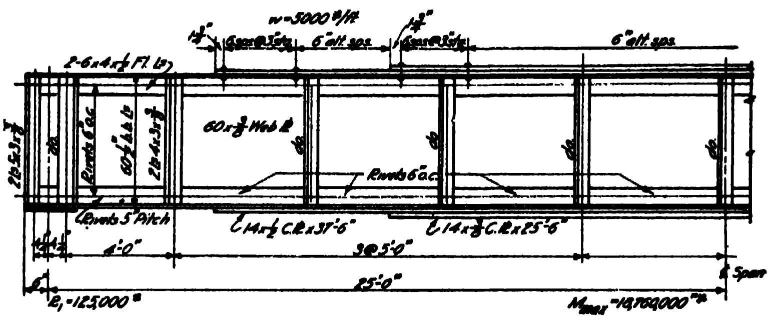


FIG. 73

Cover Plate Rivets.

$$p = \frac{R \cdot d_e \cdot A_{FN}}{V \cdot A_{CN}},\ A_{FN} = 20.81\square'',\ A_{CN} = 6.0\square'',\ d_e = 59.4.$$

First cover plate begins at $\frac{50 - 37.5}{2} = 6.25'$ from R_1

$V = 125{,}000 - 5000 \times 6.25 = 93{,}750\#$

$$V' = V \times \frac{A_{FN}}{A_{FN} + \frac{1}{8} A_W} = 93{,}750 \times \frac{13.5}{16.31} = 77{,}500\#$$

Single shear $\frac{7}{8}''$ rivet = 7220 = R 2 rows of rivets

$$p = \frac{2 \times 7220 \times 59.4 \times 20.81}{77{,}500 \times 6.0} = 38.6'' \quad \text{Too large.}$$

Tensile strength of plate = $6.0 \times 16{,}000 = 96{,}000\#$

$\frac{96{,}000}{7220} = 13+$ Use 14 rivets (7 each side).

Space rivets staggered with flange rivets $1\frac{3}{4}''$ from end, 6 spaces at 3", then at 6" alternate spaces for each cover plate.

Rivets in End Stiffeners.

$R_1 = 125{,}000\#$ $\frac{125{,}000}{7880} = 15+$ say 16 rivets

$\frac{16}{2} = 8$ in each pair of angles

$55\frac{1}{2} + 5 = 10''+$ Space 6" o.c.
Space same for intermediate stiffeners.

Web Splice.

$$M_W = M \times \frac{\frac{1}{8} A_W}{A_{FN} + \frac{1}{8} A_W} = \frac{18{,}760{,}000 \times 2.81}{20.81} = 2{,}530{,}000''\#$$

$V = 60 \times \frac{3}{8} \times \frac{3}{4} \times 12{,}000 = 202{,}000\#$
Depth available = 52" (Fig. 74 (*a*)).

Maximum allowable stress = $\frac{16{,}000 \times 26}{30} = 13{,}860\#/\square''$

Value of $\frac{7}{8}$ rivet 1" from N.A. = $\frac{7880}{30} = 263\#$

$(3)^2 = 9$
$(6)^2 = 64$
$(9)^2 = 81$
$(12)^2 = 144$
$(15)^2 = 225$
$(18)^2 = 324$
$(21)^2 = 441$
$(24)^2 = 576$

$d_n = 24''$

$1836 \times 2 = 3672 = 2\,\Sigma d^2$ for one whole line
$263 \times 3672 = 966{,}000''\#$ moment of resistance of 1 line

$\frac{2{,}530{,}000}{966{,}000} = 2.86.$ Use 3 vertical lines (51 rivets each side) $I_p = 3 \times 3672$.

$$r = \sqrt{\left(\frac{225{,}000}{54}\right)^2 + \left(\frac{2{,}530{,}000 \times 24}{3 \times 3672}\right)^2}$$

$r = 6850\#$ maximum stress on extreme rivet
$r = 7880\#$ allowable
Splice O.K

Maximum length of plate = 33'-0"
Hence splice 17'-0" from R_1.

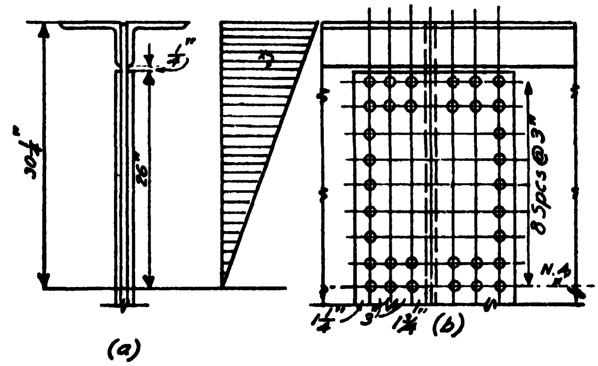


FIG. 74

Cover Plate Splice.

Maximum length of plate 33'-0". Hence splice of inside cover plate required. Theoretically 14 × ½ splice plate required. Use top cover plate as splice plate which is only 14 × ⅜. See Fig. 75.

37'-6"	37'-6"	2|12'-0"
33'-0"	25'-6"	6'-0"
4'-6"	12'-0"	4'-6"
		1'-6" distance from end of top plate to C.L. splice.

Hence run top plate over to make the splice.

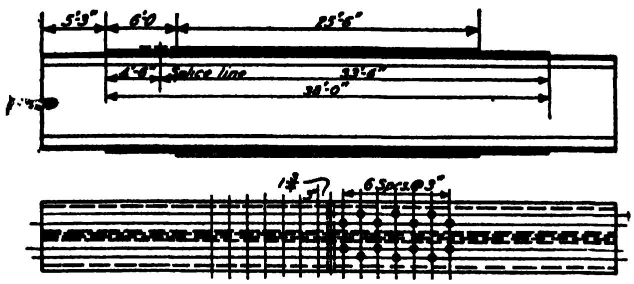


FIG. 75

No Flange Angle splice required (60'-0" lengths of angles are obtainable).

End bearing.

$R_1 = 125{,}000\#$ Allowable bearing $= 600\#/\square''$.

$$\frac{125{,}000}{600} = 208.3\square'' \text{ required.} \qquad 14'' \text{ bearing.}$$

$$\frac{208.3}{14} = 15'' \qquad \text{Use } 14'' \times 16'' \text{ bearing.}$$

$$\text{Actual pressure} = \frac{125{,}000}{14 \times 16} = 560\#/\square''$$

Projection beyond toe of fillet (Fig. 76) = 8 − 1.18 = 6.82"

$$M \text{ on } 1'' \text{ strip} = \frac{560 \times (6.82)^2}{2} = 13{,}000''\#$$

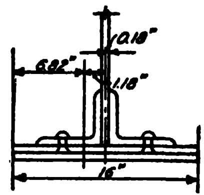


FIG. 76

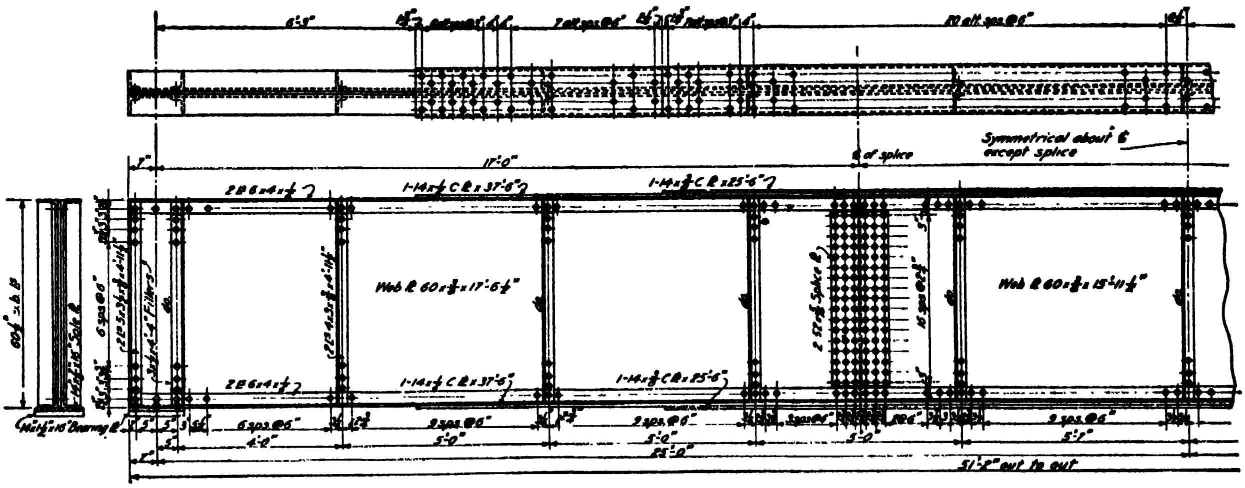


FIG. 77

M at C.L. splice $= M$ at (5'-3" + 4'-6" = 9'-9")

$$M \text{ at } 9.75' \text{ from } R_1 = \left(125{,}000 \times 9.75 - \frac{5000 \times (9.75)^2}{2}\right) 12$$

$$= 11{,}300{,}000''\#$$

$$\frac{M}{d_e} = F = \frac{11{,}300{,}000}{59.4} = 190{,}000\# \quad \frac{190{,}000}{16{,}000} = 11.8\square''$$

Flange area required at center line of splice.

⅛ A_w =	2.81
2 Ls =	7.50
14 × ⅜ Pl. =	4.50
	14.81□" available. O.K.

⅜" Pl. O.K. as splice plate.

Net area 14 × ½ Pl. = 6.0□" $\quad 6.0 \times 16{,}000 = 96{,}000\#$

$\frac{96{,}000}{7220} = 13+$ say 14 rivets

Run 14 × ⅜ plate over at left-hand end to accommodate 14 rivets beyond the center line of splice.

Try ⅞" sole plate. ⅞ + ½ = 1⅜"

$$M_r = \frac{s \times 1 \times t^2}{6} = \frac{20{,}000 \times 1 \times (1.38)^2}{6} = 6360''\#$$

13,000 − 6360 = 6640"# to be carried by wall plate

$$6640 = \frac{20{,}000 \times 1 \times t^2}{6}, \qquad t = 1.4''$$

Use 14 × ⅞ × 1'-4" Sole Plate
14 × 1½ × 1'-4" Wall Plate.

$$\frac{V}{d_e} = \frac{125{,}000}{58.52} = 2130\#/\text{inch.}$$

2130 × 14 = 29,900#

$\frac{29{,}900}{7220} = 4+$ Use 6 countersunk rivets to fasten sole plate

Use 3 × ½ × 4'-4" fillers under all stiffeners.

Figure 77 shows a detail of the girder.

40. Practical Limits for the Span.

The length of span should be taken as the distance center to center of bearings when resting on walls or running over the tops of columns, and as end to end of girder when framed between columns. Rarely are plate girders considered for spans greater than 100′-0″. For spans from 100′-0″ to 140′-0″, latticed girders are commonly used, and for spans greater than 140′-0″, some form of pin connected truss may be employed. In building construction, such spans are not generally encountered, and if so, it would probably be more economical to revise the framing so that such spans did not exist. Girders should preferably be made as single units. Occasionally, girders are fabricated in two sections for convenience of shipment, and then the sections are riveted together at the site before erection. If avoidable, this should not be done.

41. Limiting Depths of Girders.

The maximum depth of plate girders is established by overhead shipping clearance, which is usually about 10′-6″. Rarely are girders fabricated the depth of which exceeds this figure, and ordinarily the depths would not approach it, except in special cases where a girder extended a full story height. Many formulas are given for an economic depth of girder, but seldom is one accurate for all conditions.* In general, an economical depth lies between 1/7 L for short spans and 1/12 L for long spans.

SPECIFICATION CLAUSE

Depth The depth of the girder shall not be less than one-twelfth of the span. If a shallower section than this has to be used, the section shall be increased so that its maximum deflection is not greater than if the above limiting ratios had been used.

In building work, however, the **requirements of headroom more often determine the depth,** but plans should be made to allow at least 1″ for every foot of span. Many empirical formulas have been evolved to aid in selecting plate girder depths. These should be used with caution as they are generally developed for particular cases. A common formula of this kind which is fairly general is

$$d = \frac{L}{0.005\,L + 0.543}, \qquad (S\text{–}17)$$

in which d = the approximate depth of the girder in inches, and
L = the span of the girder in feet.

42. The Weight of the Girder.

It is desirable to make an approximation for the weight of a plate girder, and its haunch if used, when the designer is calculating the load that the girder must carry. Although the weight may not affect the stresses more than 1 or 2% in some cases, in others it may be an appreciable amount, especially when the girder is to be encased in fire-resisting materials. Therefore, it is good practice to make an allowance for the weight in any case. An interesting feature in the connection of keeping the dead weight as small as possible is illustrated by the sections in Fig. 78. Other schemes may be used, of course, the natures of which depend upon the architectural requirements of the ceiling into which the girder projects.

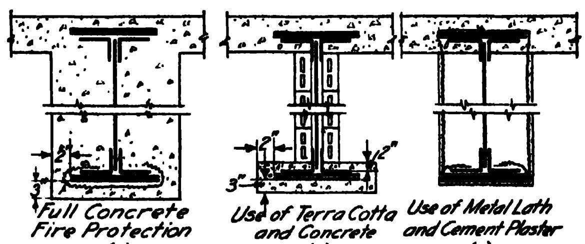


FIG. 78

Some formulas for weight may fit the cases for which they were derived, but they may not be general for all conditions. A good formula to use in making up a guide weight per square foot to use in design is

$$w = 3 + \frac{L}{4.25},\dagger \qquad (S\text{–}18)$$

in which w = the weight of the girder expressed as pounds per square foot of floor area, and
L = span of the girder in feet.

Another good rule of thumb is:

"To obtain an approximate weight of girder, multiply the area of the center cross-section in square inches by four times the length of the girder in feet."‡

This rule can, of course, only be used when the size has been approximated. Many handbooks also give the weight per foot of various tabulated sections.

Prob. 42a. Determine the approximate weight of a plate girder to span 60′-0″, in pounds per square foot of floor area. Use formula S–18.

* Some designers state that an economic depth results when the amount of material in the flange equals the amount of material in the web, if no cover plates are considered.

If the depth is increased, less flange material is required, but the thickness of the web and the material in the stiffeners is increased.

A depth less than 3′-0″ is awkward to design, but in special cases depths of 2′-0″ have been used.

† Boston Bridge Works, Inc.

‡ Article by R. Fleming, Engineering News Record, June 3, 1920.

43. Engineering Drawings and Structural Details.

With the architect's general plans and such sections and details as are available for a basis, the engineer evolves a sketch framer upon which is indicated the distribution of the steel, which in most cases is an exercise of judgment, and upon which he bases his design sheets. The beams are marked in a consecutive order by numbers so as to establish a definite relation between his sketch and the design sheets. Such sketches may be numerous and they are used by the steel draftsman to lay out the framing drawings which are incorporated in the contract set for proposals.

When showing the results of his girder design on the framing plan, the engineer usually gives only the main features of the girder as compared with the details that eventually will be required for its fabrication. The latter are usually designed in the fabricator's office. The engineer should define the span, either center to center of walls or face to face of columns, the projection beyond the center line of bearing, the depth back to back of flange angles, the sizes of web plate, flange angles, and cover plates, the lengths of cover plates, the loads and the reactions and where they are located. Figure 72 shows such an engineer's drawing.

Some engineers, in order to assure a proper interpretation of the design, prefer to give additional information as an aid in obtaining proper relations of the secondary parts. Such data would be items like sizes of stiffener angles at reactions, those at concentrated loads, sizes and spacing of intermediate stiffeners, the rivet pitches required in each panel, bearing plate sizes, and where splices, if used, should be located. Figure 73 shows such a drawing. These data should in any event be incorporated in the engineer's design sheets for purposes of comparison with the fabricator's details.

Any details such as those mentioned in the above paragraphs, if not supplied by the engineer, must be designed by the fabricator. In addition, the details of the rivets in stiffeners, flange angles, and cover plates, splices, sole plates and so on, must be prepared. Figure 77 gives a shop drawing of the same girder shown in Figs. 72 and 73. A good structural designer should be familiar with the design of all the details involved in plate girder fabrication, as he will be better able to call for practical combinations of the main members, although he may not be regularly called upon to design the details. Furthermore, such knowledge is invaluable when checking drawings which have been submitted by the fabricator for approval.*

SECTION 5B

THE WEB PLATE

44. Requirements.

The function of the web plate is to provide against the shear caused by the loads and to tie the flanges together so that the tension and compression may form the proper resisting couple. When the depth of the girder has been decided upon, as has been previously explained (Art. 41), the first step in the design is to determine the size of the web plate.

The **depth** of the plate should be in multiples of 2″ and preferably **in multiples of 6″** in order to match stock plate sizes. Girders are sometimes described nominally by the depth of the web plate, such as "a 36″ girder," or "a 48″ girder." But in reality the angles should project beyond the web plate to take care of any irregularities in the plate due to the shearing process. If the angles were set flush with the edge of the plate, it might mean considerable chipping before any cover plates could be riveted to the angles. Setting the angles out also gives a more finished appearance to the fabrication. If there were no cover plates, however, and the girder were exposed to the weather, rain pockets would exist in the top flange, which would cause rusting. This space should be covered or at least filled with pitch if cover plates are omitted. For this reason, some engineers prefer to run the first cover plate the full length of the girder. The standard practice is to set the angles out ¼″ top and bottom, thus making the **distance back to back of angles** (b.b. ∟s) ½″ **greater than the depth of the web plate** (Fig. 79).

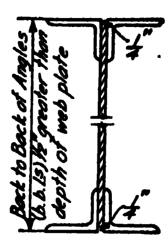


FIG. 79

A **minimum thickness** of web plate is established because rusting would have a relatively dangerous effect on a very thin plate, and also because the ordinary conditions of shear would cause abnormal tendencies to buckle.†

* In consequence, this chapter includes a treatment of the detail design features as well as those of the design of the main parts. Sections 5A, 5B, 5C, 5D and 5E deal principally with the latter, while the remaining sections are based, in general, upon the sections preceding them.

† For very light girders, ¼″ is occasionally used, but the bearing value of the rivets is low and this unduly increases the expense. Some designers use a minimum of ⅜″ in all cases.

SPECIFICATION CLAUSES

Web Plate Thickness The minimum thickness of web plate shall be $\frac{5}{16}$". When the girder is exposed to climatic conditions, the minimum shall be of $\frac{3}{8}$".

The thickness of web plate shall not be less than 1/160 of the unsupported distance between the flange angles.

The last clause is given to prevent extremely deep and thin webs. Since the shear diminishes along the span, it is not necessary theoretically to have the web of uniform thickness. However, the cost of the fabrication of an exacting design is more than offset by the cost of the extra material which a uniform thickness would entail. The two cases are illustrated in longitudinal sections in Fig. 80.

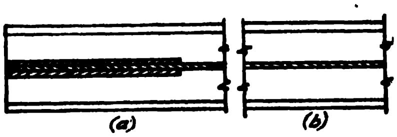


FIG. 80

45. The Design of the Web Plate.

As has been stated, the purpose of the web is to resist the transverse shear. The value of the maximum external shear, V, ordinarily the maximum end reaction, must be known in order that the plate may be properly designed. This shear must be carried at a safe working stress. In general mechanics it is shown that the intensity of vertical shear is equal to the intensity of horizontal shear. The horizontal shear varies from 0 at the extreme fibre to a maximum at the neutral axis and the vertical shear is equal to it. Figure 81 shows this variation.

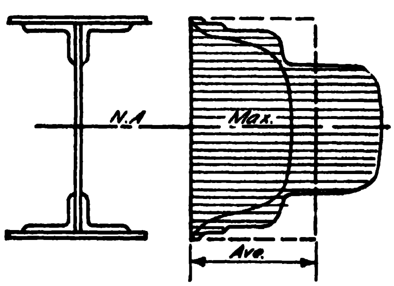


FIG. 81

The intensity of horizontal shear is exactly expressed as

$$q = \frac{V \cdot Q}{b \cdot I} \text{ (Art. 12).}$$

To use the formula given would be the exact procedure, but this would be laborious and inconvenient, and the additional accuracy obtained would scarcely be offset by the additional calculations necessary. Q is the only variable in the above formula for a given section of a girder and its magnitude is proportional to the distance of the metal from the neutral axis. But these values of Q (the statical moment) would change more slowly than the corresponding distances from the neutral axis. To offset this, the **shear is assumed as uniformly distributed over the effective cross-section** of the web, or

$$v = \frac{V}{A_W} \text{ (Art. 12).}$$

There is considerable discussion as to whether the effective cross-section should be taken as the gross area or the net area of the web. There certainly will be some rivets driven through the web, especially at supports where stiffeners are used. Some authorities claim that the rivets fill the holes since they are driven hot and under pressure, making the gross section effective. This does not allow for possible loose rivets.

An interesting theory is that shown by Mr. R. Fleming.* In Fig. 82, let a–b be a section through an end stiffener where the shear is a maximum. The rivets relieve the shear in the web plate by one-half their bearing value, thereby reducing the shear in the net section by this amount. The section through rivets on line c–d equals one-half of 1.73 of that on line a–b, but the shear is reduced by only one-quarter of the bearing value of the rivet.

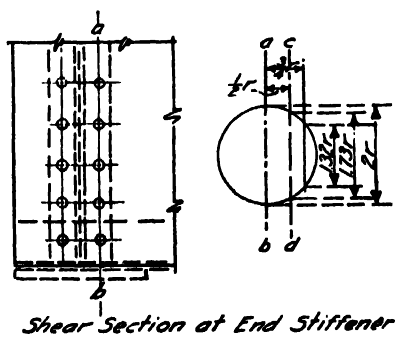


FIG. 82

The above opinion would tend to show that the gross section might be used, but as before, no allowance is made for loose rivets and also the punching of the web should be restricted. An important point is that the flange has a vital influence on the distribution of shear, as indicated in Fig. 81. The exact value can be calculated only by the formula given, while the use of the gross area as the effective area produces an error which decreases the factor of safety. Using the net area also produces an error, but this is on the safe side. Accordingly, the authors recommend using the **net area as the effective area** of the web. This is advisable since

* Engineering News Record, June, 1923.

the designer usually does not control the depth. The formula may then be expressed as

$$v = \frac{V}{A_{WN}}, \text{ in which } (S\text{–}19)$$

V = the maximum external vertical shear in #,
A_{WN} = the net area of the web plate in □″, and
v = the average intensity of shear in #/□″

The common assumption is to consider the **net area of the web as three-quarters of the gross area.** A representative case corresponding to this would be the use of 1″ rivet holes (theoretical allowance for ⅞″ rivets) 4″ o.c., for the depth of the web plate; deducting the holes to get the net section, — 1″ out of every 4″ would be lost. This case is not far from the average where stiffeners are used at end supports or where the shear is heavy, such as at floor beams or at column connections.

Prof. C. M. Spofford in his " Theory of Structures "* gives several illustrative examples showing the variation produced in the shear by the use of $q = \frac{V \cdot Q}{b \cdot I}$ and that by assuming the shear uniformly distributed on the net area. These cases show that the error introduced by using the approximate method is usually less than 10%, and on the safe side.

In view of many tests which have been made, it is not unreasonable to use **12,000#/□″** for the **shearing stress** on the net section.†

Illustrative Prob. 45a. What size of web plate should be used if the girder is limited to a depth of 4′-0″±? The maximum vertical shear = 130,000#. Allowable shearing stress 12,000#/□″ on net section.

$$\text{Net area required} = A_{WN} = \frac{V}{v} = \frac{130{,}000}{12{,}000} = 10.82\square''$$

$$\text{Gross area} = A_W = \tfrac{4}{3} A_{WN} = \tfrac{4}{3} \times 10.82 = 14.43\square''$$

$$\text{Thickness} = t = \frac{A_W}{h} = \frac{14.43}{48} = 0.30''$$

Try 48 × $\frac{5}{16}$ Pl.

In order to obtain a reasonable spacing of rivets in the flange angles, it may become necessary to increase the thickness of the web plate, as calculated in the above illustrative problem. This subsequent change is often required in order to obtain an economical bearing value for the rivets. Also a more economical arrangement of stiffeners may be obtained in certain cases if the web thickness is increased (Art. 59).

Illustrative Prob. 45b. Determine the size of web plate for a maximum vertical shear of 200,000#, depth limited to 5′-0″±. Use 12,000#/□″ for the allowable stress. L = 50′-0″.

$$d = \frac{50}{.005 \times 60 + 0.543} = 59'' +$$

$\frac{1}{10} L = 60''$ Use 60″ web plate.

$A_{WN} = \frac{200{,}000}{12{,}000} = 16.7\square''$ $\quad \frac{4}{3} \times 16.7 = 22.2\square''$

$\frac{22.2}{60} = 0.37''$ Use 60 × ⅜ web plate.

Prob. 45c. Select a web plate for a plate girder to span 70′-0″ and carry a maximum vertical shear of 300,000#.

SECTION 5C

THE FLANGES

46. Methods Employed in the Design.

Heretofore in the design of steel beams, the flexural formula $M = \frac{s \cdot I}{c}$ has been used. For plate girder work this formula is not deemed by all to be the most convenient form to use and it can be varied in application, but still embodying the same principles. There are various methods of such design used as enumerated below.

MOMENT OF INERTIA METHODS

(1) The **moment of inertia of the gross section**‡ may be used in the above formula. There is no practicable objection, but the method is not theoretically correct. The reason for this is that both flanges are usually made the same in cross-section while they do not have the same strength, one being controlled by buckling, the other by its net section resisting tension.

(2) The **moment of inertia of the net section** may be used instead of (1). This method is generally considered too severe and indirect.

(3) Using a **combination** of (1) and (2), the moment of inertia of the cross-section may be based upon the **gross area of the top flange** and the **net area of the bottom flange.** A more exact result than in either of the first two methods is obtained, but the calculations are quite involved.

FLANGE AREA METHODS

(4) The bending moment is assumed to be carried entirely by the flange proper.§ This procedure is sometimes called the **chord stress**

* 1st. Edition. John Wiley & Sons, Inc.
† Also the specification of the American Society of Civil Engineers, 1923.
‡ Used by the American Bridge Co. for the design of plate girders in buildings.
§ This method of design is required by the New York City Building Code.

method, and it is relatively simple in application.

(5) Method (4) may be used, except that **one-eighth** of the **gross area of the web** is considered **available as flange area.***

The first three methods are more accurate than the last two mentioned, but they are more laborious to use, and are usually replaced by a flange area method which is slightly less accurate but much more convenient and direct in its solution. The main point of distinction between the two major groups is that the first deals directly with the fibre action, while the second assumes that the tension and compression are uniformly distributed over the respective flanges. There are limitations of the flange area method, however, where the error is reasonably small. Shallow girders with several cover plates should be designed by the moment of inertia method as other methods will give flange areas which are too small.

Mr. R. Fleming† gives an interesting table of comparison for the moments of resistance of various plate girder sections calculated according to different methods, showing that the percentages of difference are not large. Prof. C. M. Spofford‡ gives several illustrative problems showing the comparative results obtained by the use of $M = \frac{s \cdot I}{c}$ and the flange area method for representative types of girders. The errors given in these problems vary from $\frac{1}{2}$ of 1% to 4%, averaging about 2%, on the safe side for the flange area method.

The method of using $\frac{1}{8}$ **the gross area of web** as available flange area seems to be the most popular, and the authors recommend it as the **most reasonable** and feasible to use. From Prof. Spofford's problems just mentioned, it seems that this method makes the best approximation to the exact method. It seems reasonable that the portion of the web confined between the flange angles should be counted upon as resisting some bending. However, some designers prefer to cut the $\frac{1}{8}$ web allowance to $\frac{1}{10}$ or $\frac{1}{12}$.

After the flange section has been definitely established, the moment of inertia may be calculated and the extreme fibre stress found by an application of $M = \frac{s \cdot I}{c}$. This stress should be within the allowable limit.

47. The Flange Area Formula.

The theoretical formula based upon the flange area required may be derived in the following manner:

* This method is allowed in the Boston Building Code.
† Engineering News Record, June, 1923.
‡ "Theory of Structures," 1st Edition — John Wiley & Sons, Inc.

Fig. 83, let

I = the total moment of inertia of the whole gross section about the neutral axis.
I_F = the moment of inertia of each gross flange area (A_F) about its own center of gravity.
I_W = the moment of inertia of the gross web area (A_W) about the neutral axis.
d_e = the effective depth of the girder, or the distance between the centers of gravity of the flange areas.
t = the thickness of the web in inches and,
h = the depth of the web plate in inches.

From the basic definition of moment of inertia in mechanics, $I_0 = I + \Sigma a \cdot d^2$,

$$I = 2\,I_F + 2\,A_F \cdot \left(\frac{d_e}{2}\right)^2 + I_W$$

$$I = 2\,I_F + \frac{A_F \cdot d_e^2}{2} + \frac{t \cdot h^3}{12}.$$

The moment of inertia of the flange about its own center of gravity axis is very small compared with the other moments of inertia and may be neglected without undue error. In any case this error is on the safe side. Then

$$I = \frac{A_F \cdot d_e^2}{2} + \frac{t \cdot h^3}{12}.$$

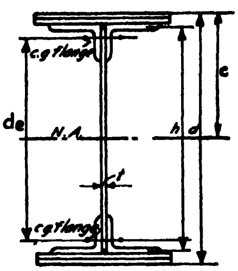


Fig. 83

Let d = the distance out to out of extreme fibres, and
c = the distance from the neutral axis to the extreme fibre.

$$\frac{I}{c} = \frac{\frac{A_F \cdot d_e^2}{2} + \frac{t \cdot h^3}{12}}{\frac{d}{2}} = \frac{A_F \cdot d_e^2}{d} + \frac{t \cdot h^3}{6\,d}.$$

But $\frac{I}{c} = \frac{M}{s}$ from the flexural formula.

Substituting, and solving for A_F,

$$A_F = \frac{M \cdot d}{s \cdot d_e^2} - \frac{t \cdot h^3 \cdot d}{6 \cdot d \cdot d_e^2}.$$

Expressed differently,

$$A_F = \frac{M}{s \cdot d_e}\left(\frac{d}{d_e}\right) - \frac{t \cdot h}{6}\left(\frac{h}{d_e}\right)^2.$$

The value of d_e is seldom greater than h, and is usually smaller, as the cover plates are generally cut off before the end of the girder is reached. A

common specification already given (Art. 37) that d_e should not be greater than the distance back to back of flange angles. Hence $\frac{h}{d_e}$ is assumed as 1. This will make the last term of the equation above slightly less than its true value, but since this term is subtracted, the error introduced is on the side of safety, and in any instance the latter term is small compared with the first term. The ratio $\frac{d}{d_e}$ is also assumed as unity. The value of d_e will always be less than d, so that this assumption will introduce an error which is on the unsafe side. This error is larger for shallow girders with heavy flanges, hence the reason for using $M = \frac{s \cdot I}{c}$ in such cases, as explained in Art. 46. This and the previous error would tend to be compensating.

With the foregoing assumptions, the equation reduces to

$$A_F = \frac{M}{s \cdot d_e} - \frac{t \cdot h}{6}.$$

The last term $\frac{t \cdot h}{6}$, however, is a fraction of the gross area of the web. To carry out the assumption of the net area of the web as in Art. 45, $\frac{3}{4}$ the gross area is assumed as the net area. Hence,

$$\tfrac{3}{4} \times \frac{t \cdot h}{6} = \frac{t \cdot h}{8}$$

and

$$A_F = \frac{M}{s \cdot d_e} - \frac{t \cdot h}{8}.$$

Tension cannot be transmitted through rivet holes and the net section must, therefore, be used, as the tension flange then controls. If A_{FN} = the net area of the flange, the formula finally becomes:

$$A_{FN} = \frac{M}{s \cdot d_e} - \frac{t \cdot h}{8}. \qquad (S\text{–}20)$$

This replaces A_F by A_{FN} and disregards the incident movement of the neutral axis. The section is considered solid, and the neutral axis one-half way between the top and the bottom of the girder, but the holes in the tension flange (if considered only) tend to shift the exact location of the neutral axis up from the center, diminishing the value of I, and increasing the distance from the neutral axis to the extreme fibre. This helps to overcome the error introduced by the assumption, $\frac{d_e}{d} = 1$. Test calculations have shown that for a section through a maximum number of rivets, the neutral axis moves up only slightly, usually less than $\frac{1}{2}''$. The net section also varies in the length of the girder. The neutral axis does not move up and down in actuality, but remains approximately near the neutral plane of the whole.

Reviewing the above formula, the important point is that the stress F is assumed as uniformly distributed over the whole flange area, and that the resultant is assumed to act at the center of gravity of the gross area of each flange. The distribution of stress has not been completely defined to give absolute proof of the above. The allowable average unit stress is taken as that specified for the extreme fibre, namely 16,000#/□″.

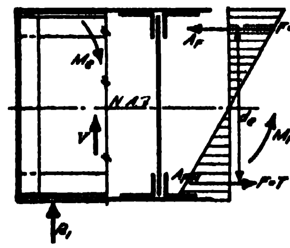


FIG. 84

Referring to Fig. 84, the external bending moment due to the loading is M_e. The resisting moment on the above basis is

$$M_e = M_r = F \cdot d_e, \quad \text{or}$$

$$F = \frac{M}{d_e} = \text{the flange stress}$$

$$A_{FN} = \frac{F}{\text{allowable tensile stress}}. \qquad (S\text{–}21)$$

The external bending moment can be obtained from the loading diagram. The dimensions of the web plate, t and h, are established from the shear calculations (Art. 45). Since the makeup of the flange is not established at this time, the correct value of d_e is not known, and a trial value of d_e must be used. The best approximation to the exact value is to assume the effective depth equal to the depth of the web plate.* With such data, A_{FN} can be found and the makeup of the flange determined. The assumption will give a flange area slightly too large, and in selecting the makeup of the flange, this may be considered.

Illustrative Prob. 47a. The maximum bending moment = 1,250,000″#. Web plate 48 × $\frac{3}{8}$. Tensile stress = 16,000#/□″. Find the required net flange area. $\frac{1}{8}$ A_W available.

$$F = \frac{M_e}{d_e} = \frac{1{,}250{,}000 \times 12}{48} = 312{,}500\#\ \text{flange stress.}$$

$$\text{Area required} = \frac{312{,}500}{16{,}000} = 19.53\square''$$

$$\tfrac{1}{8}\ \text{gross area web} = 48 \times \tfrac{3}{8} \times \tfrac{1}{8} = \underline{\ 2.25\ }$$

$$A_{FN} = 17.28\square''.$$

If no allowance had been made for the web plate, 19.53□″ would have to be used for the flange section proper.

* F. A. Dufour uses the following assumption: b. to b. of ∟s − 2″ = d

It becomes necessary to make **allowance for the rivet holes** in the tension flange. There is no hard and fast rule for doing this, but a reasonable approximation may be made. It is seldom necessary to take more than three holes out of any flange angle or cover plate, and often the number may be less, as all the holes are not apt to occur at the same section. However, it is necessary to consider whether a zigzag section through the holes will give a less net area (Art. 48). In any case, a liberal allowance should be made so that the design will not have to be revised when a close spacing of rivets is required. Since it is more or less confusing to determine how many holes to take out to get the net section, the following rules of thumb are given as a guide (Fig. 85):

(1) Single line of rivets or two lines of rivets staggered, (*a*) or (*b*) — each angle — 1 hole out.

(2) If connections to angle occur (such as lateral bracing) — each angle — $1\frac{1}{2}$ holes out (*c*).

(3) Flange angles with cover plates, if one row in each leg of angle — each angle — 1 hole out (*d*).

(4) Flange angles with cover plates, if two rows in each leg of angle (*e*) — each angle — 2 holes out.

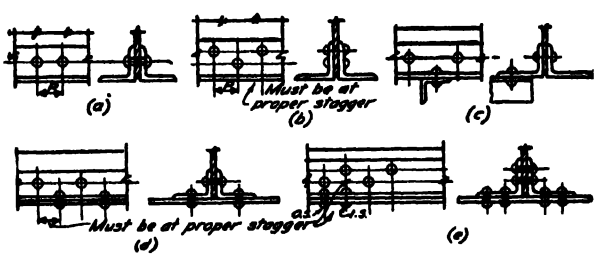


FIG. 85

The diameter of rivet to be used is fixed by practical considerations, that is, to get a rivet which shall have a strength in proportion to the size of girder. Obviously the smaller the rivets, the closer their spacing to develop the strength of the parts connected. On the other hand, the larger the rivet, the more the area which must be deducted to obtain the net section. The lower limit of size is somewhat governed by the fact that the spacing should not be less than three diameters of the rivet, and the upper limit by the maximum spacing of 6″. The cost of punching also increases with the size of the holes. The size of **rivets** generally employed is $\frac{3}{4}''$ **for the smaller girders, and $\frac{7}{8}''$ for the larger girders,** with 1″ rivets for extreme cases.

Prob. 47b. Determine the net section required for a maximum bending moment of 2,500,000″#. Tensile stress = 16,000#/□″. Web plate 60 × $\frac{5}{16}$. No allowance for web plate.

Minimum Stagger of Rivets to Maintain Net Section.

When rivets are driven in two or more lines, and it is desirable to keep the net section a maximum, the rivets must stagger* a sufficient distance so that only the minimum number of holes need be taken out. Past experiments tended to show that a failure would not occur on a zigzag line which had a net area larger than 30% of the section across the member. Recent experiments show that the probability of rupture is the same in either of the directions shown in Fig. 86 (*a*) when the net areas are equal. The old conclusion has been abandoned in modern practice and the minimum stagger is based on making the distances equal. Consider the angle in Fig. 86 (*b*) where one leg is developed into the

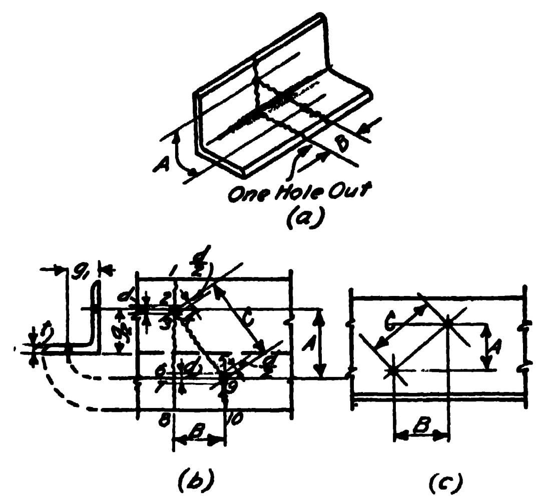


FIG. 86

plane of the other so that A equals the sum of the two gauges minus the thickness of the angle. This also corresponds to a plate with two lines of rivets in it, a distance A apart. The steel may rupture along the line 1–2–3–6–7–8, or along the line 1–2–4–5–9–10. These distances must be equal if the resistance along each line is to be the same. Let d equal the diameter of the rivet plus $\frac{1}{8}''$ (the usual allowance for net section). Then

1–2 = 1–2 and 7–8 = 9–10 by inspection.

Hence 3–7 must equal 4–5.

$$3\text{–}7 = A \quad \text{and} \quad 4\text{–}5 = C - d. \quad \text{But } C = \sqrt{A^2 + B^2}.$$

$$\therefore 4\text{–}5 = \sqrt{A^2 + B^2} - d = 3\text{–}7 = A,$$

or

$$A = \sqrt{A^2 + B^2} - d$$

$$A + d = \sqrt{A^2 + B^2}.$$

* Sometimes called chain riveting.

Squaring both sides,

$$A^2 + 2\,A \cdot d + d^2 = A^2 + B^2, \quad \text{or}$$

$$B = \sqrt{2\,A \cdot d + d^2} = \text{Minimum stagger} \qquad (S\text{–}22)$$

(One hole out).

This also applies to Fig. 86 (c) for rivets in two gauge lines of one leg only. The distance C, however, must conform to the requirements of minimum center to center distances.

Illustrative Prob. 48a. In a $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$ angle ($2\frac{1}{4}''$ gauges) what distance can B be made to maintain a net section of one hole out for $\frac{3}{4}''$ rivets?

$$2\tfrac{1}{4} + 2\tfrac{1}{4} - \tfrac{1}{2} = 4'' = A \qquad d = \tfrac{3}{4} + \tfrac{1}{8} = \tfrac{7}{8}''$$

$$B = \sqrt{2\,A \cdot d + d^2} = \sqrt{2 \times 4 \times \tfrac{7}{8} + (\tfrac{7}{8})^2} = 2\tfrac{13}{16}''.$$

If the holes in the opposite gauge lines of such an angle were closer than $2\frac{13}{16}''$ on centers, the net section is determined by deducting two holes instead of one. If $B = 2\frac{13}{16}''$ or more, only one hole need be deducted.

Substituting

$$A - d = \sqrt{A^2 + B^2} - 2\,d,$$

or

$$A + d = \sqrt{A^2 + B^2}.$$

Squaring both sides,

$$A^2 + 2\,A \cdot d + d^2 = A^2 + B^2,$$

or

$$B = \sqrt{2\,A \cdot d + d^2} \quad \text{(as before).}$$

The cases shown in Fig. 87 (c) are similar and the same formula may be employed. It may also be used for plates, when A is the distance between the rivet lines. Table 26 gives values of B for varying values of A, based upon the above formula. For $\frac{5}{8}''$ rivets, the values given for $\frac{3}{4}''$ rivets may be reduced $\frac{1}{8}''$. The values for $1''$ rivets may be obtained by adding $\frac{1}{8}''$ to those for $\frac{7}{8}''$ rivets.

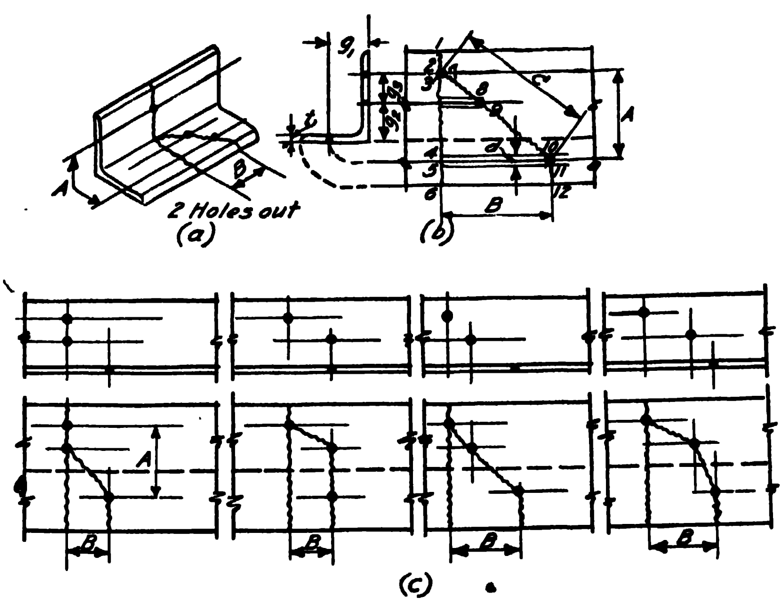


FIG. 87

TABLE 26

MINIMUM STAGGER OF RIVETS (B) TO MAINTAIN NET SECTION

Dimensions in Inches

A	$\frac{3}{4}''$ Rivet B	$\frac{7}{8}''$ Rivet B	A	$\frac{3}{4}''$ Rivet B	$\frac{7}{8}''$ Rivet B
1	$1\frac{5}{8}$	$1\frac{3}{4}$	5	$3\frac{1}{16}$	$3\frac{5}{16}$
$1\frac{1}{2}$	$1\frac{7}{8}$	2	$5\frac{1}{2}$	$3\frac{1}{4}$	$3\frac{1}{2}$
2	$2\frac{1}{16}$	$2\frac{1}{4}$	6	$3\frac{3}{8}$	$3\frac{5}{8}$
$2\frac{1}{2}$	$2\frac{1}{4}$	$2\frac{7}{16}$	$6\frac{1}{2}$	$3\frac{1}{2}$	$3\frac{3}{4}$
3	$2\frac{7}{16}$	$2\frac{5}{8}$	7	$3\frac{5}{8}$	$3\frac{7}{8}$
$3\frac{1}{2}$	$2\frac{9}{16}$	$2\frac{13}{16}$	$7\frac{1}{2}$	$3\frac{3}{4}$	4
4	$2\frac{13}{16}$	3	8	$3\frac{7}{8}$	$4\frac{1}{8}$
$4\frac{1}{2}$	$2\frac{15}{16}$	$3\frac{3}{16}$	$8\frac{1}{2}$	4	$4\frac{1}{4}$

In any case, the center to center distance between rivets in one plane must meet the minimum requirements of 3 diameters of the rivet.

Illustrative Prob. 48b. Given a $6 \times 3\frac{1}{2} \times \frac{1}{2}$ angle with gauges in the $6''$ leg $2\frac{1}{4}''$ and $2\frac{1}{2}''$, and in the $3\frac{1}{2}''$ leg $2\frac{1}{4}''$. Check the value of B in Table 26 to maintain a net section of 2-$\frac{7}{8}''$ rivet holes out. Figure 88 (a).

$$2\tfrac{1}{2} + 2\tfrac{1}{4} + 2\tfrac{1}{4} - \tfrac{1}{2} = 6\tfrac{1}{2}'' = A \qquad d = \tfrac{7}{8} + \tfrac{1}{8} = 1''$$

$$B = \sqrt{2\,A \cdot d + d^2} = \sqrt{2\,(6\tfrac{1}{2}) \times 1 - (1)^2} = 3\tfrac{3}{4}''.$$

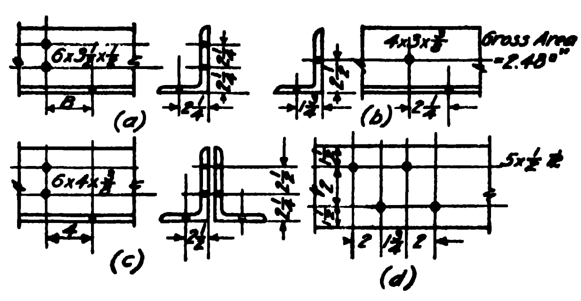


FIG. 88

Consider the angle in Fig. 87 (a) and (b), one leg of which is developed into the plane of the other, as before. Here A equals the sum of three gauges minus the thickness of the angle. The steel may rupture along the line 1–2–3–4–5–6, or along the line 1–2–7–8–9–10–11–12.

1–2 = 1–2 and 4–5 = 11–12 by inspection.

Then (3–5)–d must equal 7–10 if the zigzag line is to just maintain two holes out only.

$$A - d = C - 2\,d.$$

But

$$C = \sqrt{A^2 + B^2}.$$

Prob. 48c. What is the maximum allowable tension that the angle in Fig. 88 (*b*) can sustain at 16,000#/□"? ¾" rivets.

Prob. 48d. What is the maximum allowable tension that the pair of angles in Fig. 88 (*c*) can sustain at 16,000#/□"? ¾" rivets.

Prob. 48e. What is the net section in Fig. 88 (*d*)? Is the spacing of rivets center to center satisfactory?

49. Practical Requirements for the Flange Parts.

Some of the common combinations of angles and plates to form the flanges of girder sections are:

4 × 3 Ls — no C.Pl.	6 × 6 Ls — no C.Pl.
5 × 3½ Ls — no C.Pl.	6 × 6 Ls — 14" or 16" C.Pl.
5 × 3½ Ls — 12" C.Pl.	8 × 8 Ls — 18" or 20" C.Pls.
6 × 4 Ls — no C.Pl.	8 × 8 Ls — 2 or 4 – 6 × 4 Ls
6 × 4 Ls — 14" C.Pl.	

These follow in general the recommendations discussed in Art. 37. The maximum thickness of plates and angles should be limited by the rivets (Art. 23). Some designers set a limit of ¾" thick for any case.

SPECIFICATION CLAUSE

Minimum Thickness The minimum thickness of angles without cover plates shall be not less than 1/12 the width of the outstanding leg of the angle.

The thickness of cover plates should preferably be nearly the same as that of the flange angles. If more than one plate is used, all should be of the same thickness, or diminish in thickness from the flange angles outward. The latter is not necessary but is the usual practice. Some engineers believe that if the thinner plate is placed next the flange angles, greater economy will result, especially where it is desirable to use a full length plate. The minimum thickness should be ⅜", as thinner plates will control the number of rivets because of their low bearing value. The minimum number of plates should be used in order to avoid extra punching and handling.

The **widths of cover plates** should be kept in even inches and preferably in multiples of 2", as stock plates are nearly always carried in these widths. They should be made wide enough to project slightly beyond the toes of the flange angles but preferably not more than 2". The maximum width is sometimes limited by the architectural details of ceiling work.

SPECIFICATION CLAUSE

Width of Plates The projection beyond the outer line of rivets shall not exceed eight times the thickness of the thinnest plate, nor be greater than 5" (Distance *n* in Fig. 89). The distance between lines shall not exceed thirty times the thickness of the outer plate (Distance *m* in Fig. 89).

This specification is given to protect against the buckling of thin plates as indicated in an exaggerated way in the figure. For widths of plates exceeding the above specification, Mr. R. Fleming suggests the following*:

SPECIFICATION CLAUSE

Width of Plates (alternate) Cover plates to have their full sectional area included in flange section shall not project beyond the outer line of rivets more than 6" or more than eight times the thickness of the thinnest plate. Cover plates exceeding these limits not more than 50%, may have one-half the excess section included in flange section. No part of a projection beyond the outer line of rivets more than 9" or more than twelve times the thickness of the thinnest plate shall be considered as part of the flange section.

Illustrative Prob. 49a. Make up a flange section to supply the net area of 17.28□" from Illustrative Prob. 47 *a*. Use ¾" rivets. As a guide the flange angles should constitute approximately one-half of the flange area (Art. 37).

$\frac{1}{2} \times 17.28 = 8.64$□"

For one angle $\frac{8.64}{2} = 4.32$□"

Gross area 6 × 4 × ⅝ L = 5.86

2 holes out

⅞ × ⅝ × 2 = 1.09

4.77

2

9.54

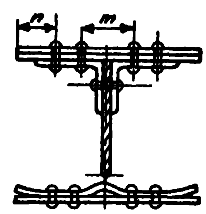


FIG. 89

Area still to be supplied = 17.28 − 9.54 = 7.64□"

Try 14" C.Pl.

Gross area 14 × ⅝ = 8.75"

2 holes ⅞ × ⅝ × 2 = 1.09

7.66" O.K. Try 2 Ls 6 × 4 × ⅝, 1 C.Pl. 14 × ⅝

In determining a section, it is good practice to make a sketch and label the sizes, allowing for the rivet holes to be deducted. This will also be useful in checking up the centers of gravity and the value of d_e. Very often structural tables are a material aid in selecting a section.

Some engineers use the gross compression flange as acting with a stress of 14,000#/□" and the net tension flange as acting with a stress of 16,000#/□". In theoretically checking the original assumption for the value of d_e (Art. 47), it is necessary to determine the neutral axis of the entire section and the centers of gravity of the two flanges, all based upon a gross compression flange area and a net tension flange area. These calculations will theoretically be as follows:

Original assumption of d_e = 48.0" (depth of web plate).

Center of Gravity of Entire Section. (Fig. 90.)

A_{CG} = 14 × 0.625 = 8.75□" (gross plate area in compression flange)

A_{AG} = 2 × 5.86 = 11.72□" (gross angle area in compression flange)

A_W = 48 × 0.375 = 18.00 (gross area of web plate)

A_{CN} = 8.75 − (2 × ⅞ × ⅝) = 7.66□" (net plate area in tension flange)

* From an article in Engineering News Record, June 14, 1923.

$A_{AN} = 11.72 - (4 \times \frac{7}{8} \times \frac{5}{8}) = 9.54$□″ (net angle area in tension flange)

A_T = total active section of girder (gross area of compression flange + gross web plate + net tension flange)

$$\Sigma M_{x\text{-}x} = 8.75 \times 0.313 + 11.72 \times 1.65 + 18.00 \times 24.87 + 7.66 \times 48.10 + 9.54 \times 49.44 = A_T \cdot y_o{}^*$$

$$y_o = \frac{1300.06}{55.67} = 23.40''$$

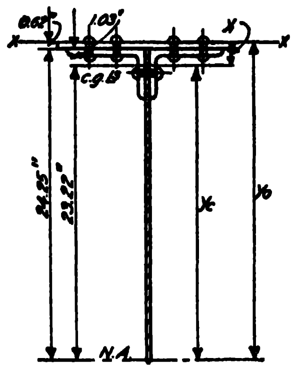


FIG. 90

Center of Gravity of Compression Flange.

$$\Sigma M_{x\text{-}x} = 8.75 \times 0.313 + 11.72 \times 1.65 = A_{FG} \cdot y_1$$

$$y_1 = \frac{22.06}{20.47} = 1.08''$$

Center of Gravity of Tension Flange.

$$\Sigma M_{x\text{-}x} = 7.66 \times 48.10 + 9.54 \times 49.44 = A_{FN} \cdot y_2$$

$$y_2 = \frac{840}{17.20} = 48.8''$$

$$d_e \text{ (actual)} = y_2 - y_1 = 48.8 - 1.08 = 47.72''$$
$$d_e \text{ (assumed)} = 48.0''$$
$$A_{FG} = 20.47 + \tfrac{1}{8} A_W = 20.47 + 2.25 = 22.72\square''$$
$$A_{FN} = 17.20 + \tfrac{1}{8} A_W = 17.20 + 2.25 = 19.45\square''$$

Resisting compression moment,

$$M_{rc} = A_{FG} \times 14{,}000 \times 47.72 = 15{,}180{,}000''\#$$

Resisting tension moment,

$$M_{rt} = A_{FN} \times 16{,}000 \times 47.72 = 14{,}860{,}000''\#$$
$$M_{max} = 1{,}250{,}000'\# = 15{,}000{,}000''\#. \quad \text{O.K.}$$

From the above analysis the following usual practice will seem reasonable. The neutral axis of the entire girder section is assumed to be at the center of gravity of the web (except when the two flanges are of unequal gross area). The center of gravity of the compression flange, based upon its gross area and exclusive of any web plate allowance is then established with relation to the neutral axis of the girder. The effective depth is then two times this distance and should be approximately equal to the assumed effective depth.

Thus for the same case as above analyzed:

$$y_c = \frac{2 \times 5.86 \times 23.22 + 14 \times 0.625 \times 24.56}{2 \times 5.86 + 14 \times 0.625} = 23.74''$$

$2\,y_c = 47.48''$,

Using the net tension flange values,

$$y_t = \frac{9.54 \times 23.22 + 7.66 \times 24.56}{17.20} = 23.78''$$

$2\,y_t = 47.56''$.

It is therefore apparent that the simple method of check for this work may be **based upon gross areas,** and when the web allowance is included,† the resulting resisting moments will compare very favorably with the maximum external moment, as follows:

$$M_e \text{ (max)} = 15{,}000{,}000''\#,$$
$$M_{rc} = 22.72 \times 14{,}000 \times 47.48 = 15{,}100{,}000''\#,$$
$$M_{rt} = 19.45 \times 16{,}000 \times 47.56 = 14{,}800{,}000''\#.$$

The question of the **accuracy to which calculations** should be **carried** again is important. The errors introduced by rounding off the figures may be considered as negligible, for they will more or less compensate each other. Therefore the following limits are suggested:

Values of d_e to the nearest 0.1″,
Values of net areas to the nearest 0.1□″.

Prob. 49b. Select the angles and cover plates necessary to furnish the net section required in Prob. 47*b*. Calculate the location of the center of gravity of the flange. Does it conform to specifications? Does the section selected supply sufficient gross area in the compression flange at 14,000#/□″?

Prob. 49c. Select the angles and cover plates necessary to furnish a net section of 38.7□″.

50. Special Considerations for the Compression Flange.

So far, the discussion of the flange area has all been referred to the tension flange mainly. Usually, however, the compression flange is made the same in section as the tension flange for the practical reasons of keeping the girder symmetrical, re-use of templates in punching holes, and so on. If it develops that such a compression flange should be increased for one reason or another, the tension flange may be increased the same amount to maintain symmetry, or special parts may be added to the compression flange only, to care for the special stresses involved, as shown in Art. 38.

It is customary to make no reduction for rivet holes in metal in compression. The assumption is that the rivets, being driven hot and under pressure, completely fill the holes. Loose rivets, hard driven rivets, and those through thick material would not completely satisfy the condition, but the assumption is practically always adopted.

If the top flange is laterally unsupported (except in cantilevers), the compression on the gross area of the flange may be excessive, as the stress is increased by sidewise bending (Art. 11). In building construction, girders are usually braced laterally within this limit by beams framing in, plank floors, concrete slabs, tie rods, and so on.

* In calculating the moment of the individual parts of the tension flange, the net section has been used, but the individual centers of gravity have been assumed as practically unchanged. The distance y_o is that to the neutral axis from the plane $x - x$ in Fig. 90.

† This area is assumed to act at the center of gravity of the flange.

SPECIFICATION CLAUSES

Limits of Unbraced Spans

Plate girders shall be stayed transversely where the unsupported length exceeds thirty times the breadth of the flange. If not stayed, the allowable compression must be reduced.

Girders Laterally Unsupported

The compression stress in girders laterally unsupported shall not exceed the value obtained by the following formulas:*

$$s_u = 16{,}000 - 200\frac{l}{b} \qquad (S\text{–}23)$$

(not to exceed 14,000) where the flange is made up of angles only, or angles and cover plates.

$s_u = 16{,}000 - 150\dfrac{l}{b}$ where the flange is made with a channel, or there is metal concentrated at the extreme fibre.

These formulas are simply a modification of the column formula $16{,}000 - 70\dfrac{l}{r}$. The term $70\dfrac{l}{r}$ is changed to $150\dfrac{l}{b}$ or to $200\dfrac{l}{b}$ (Art. 11). Some engineers prefer to keep the compression in any girder flange within $16{,}000 - 150\dfrac{l}{b}$ for reasons of safety. The constant 150 is used because the web and stiffeners help to prevent the compression flange from acting as a simple column. Another test is to figure the flange as a column of a length equal to one-half the span.

Other formulas used are:

$$\left.\begin{aligned} s_u &= 19{,}000 - 250\frac{l}{b}\dagger \text{ without cover plates}\\ s_u &= 19{,}000 - 225\frac{l}{b}\dagger \text{ with cover plates}\end{aligned}\right\}\begin{array}{l}\text{maximum}\\ 16{,}000\#/\square''.\end{array}$$

If stiffeners are used with a spacing not greater than the depth of the girder,

$$\left.\begin{aligned} s_u &= 19{,}000 - 225\frac{l}{b}\ddagger \text{ without cover plates}\\ s_u &= 19{,}000 - 200\frac{l}{b}\ddagger \text{ with cover plates}\end{aligned}\right\}\begin{array}{l}\text{maximum}\\ 16{,}000\#/\square''.\S\end{array}$$

Illustrative Prob. 50a. What is the intensity of compression in the section of Illustrative Prob. 49*a*? $s_u = 16{,}000 - 200\dfrac{l}{b}$. Is it safe? $L = 50'\text{-}0''$.

$$F = 312{,}500\# \text{ (from Prob. 47}a)$$

$$\text{gross area} = 2 \times 5.86 + 14 \times 0.62 = 20.47\square''$$

$$s_u = \frac{312{,}500}{20.47} = 15{,}250\#/\square'' \text{ actual}$$

$$s_u = 16{,}000 - \frac{200 \times 50 \times 12}{14} = 7440\#/\square'' \text{ allowable.}$$

The stress would not be safe if the girder were laterally unsupported.

Prob. 50b. If the girder in Prob. 49*b* were laterally unsupported, what gross area would be required in the compression flange? Make up a flange section. What conclusion do you draw relative to unstayed girders?

* American Railway Engineering Association. The constants 200 and 150 are used because the flange is partially braced by the web and also because the girder is not fully stressed over its entire length.

† Engineering News Record, Feb. 24, 1921.

‡ Engineering News Record, June 24, 1923.

§ The question of the compression in flanges laterally unsupported is not definitely settled, as the above range of formulas would indicate, but some provision should be made, using good judgment according to the surrounding conditions.

51. Deflection.

The deflection of a plate girder cannot usually be calculated by the formulas for ordinary beams, because the member very often has a varying moment of inertia, as the cover plates may be cut off at various points. The usual deflection formulas are based upon a constant moment of inertia, so that when the section is not constant, the method of determining the deflections must be modified.

An **approximate solution** may be made by using a moment of inertia, I, which is equal to about $\frac{3}{4}$ the average of the minimum I,|| — which occurs at the end of the girder, and the maximum I, — which occurs at the point of maximum moment, in the usual deflection formulas. If the loading is a combination of uniform and concentrated loads, the total equivalent uniform load may be used in the corresponding deflection formula for an approximate solution.

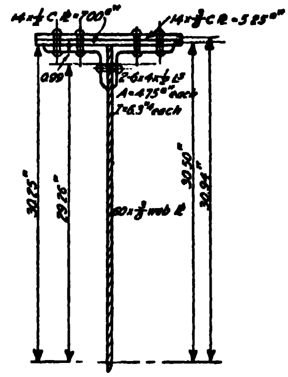


FIG. 91

Illustrative Prob. 51a. Determine the approximate value of the maximum deflection for a plate girder carrying a load of 5000#/ft. on a 50′-0″ span (see Art. 39). Web plate $60 \times \frac{3}{8}$, flange angles $6 \times 4 \times \frac{1}{2}$, one cover plate $14 \times \frac{1}{2} \times 37'\text{-}6''$, and one cover plate $14 \times \frac{3}{8} \times 25'\text{-}6''$ (Fig. 91).

$$\begin{aligned} \frac{b \cdot d^3}{12} \text{ for web plate} &= \frac{0.375 \times (60)^3}{12} &&= 6{,}750''^4\\ I \text{ for 4 } \llcorner\text{s} &= 4 \times 6.3 &&= 25\\ A \cdot d^2 \text{ for 4 } \llcorner\text{s} &= (4 \times 4.75) \times (29.26)^2 &&= \underline{16{,}300}\\ & && \ 23{,}075''^4 \end{aligned}$$

Neglect I of cover plates about their own axes.

$$\begin{aligned} A \cdot d^2 &\text{ for 2-14} \times \tfrac{1}{2} \text{ C.Pls.}\\ &= (2 \times 7.0) \times (30.50)^2 &&= 13{,}000\\ A \cdot d^2 &\text{ for 2-14} \times \tfrac{3}{8} \text{ C.Pls.}\\ &= (2 \times 5.25) \times (30.94)^2 &&= \underline{10{,}035}\\ & I \text{ (total)} &&= 46{,}110''^4 \text{ gross.} \end{aligned}$$

$$\begin{aligned} I_{\min} &= 23{,}075\\ I_{\max} &= \underline{46{,}110}\\ 2&\,|\underline{69{,}185}\\ &\ 34{,}593''^4 = I \text{ (ave.)} \end{aligned} \qquad \tfrac{3}{4} \times 34{,}593 = 25{,}930''^4$$

|| The constant $\frac{3}{4}$ is an approximation to allow for the difference between straight line variation and that for the usual moment curve (inverse proportion).

$$D = \frac{5\,W \cdot l^3}{384\,E \cdot I} = \frac{5 \times (5000 \times 50) \times 50 \times 12 \times 50 \times 12 \times 50 \times 12}{384 \times 30{,}000{,}000 \times 25{,}930}$$

$$D = 0.91''$$

$$D \text{ (allowable)} = \frac{l}{360} = \frac{50 \times 12}{360} = 1.66''. \quad \text{O.K.}$$

A **more exact solution** may be obtained by using a graphical method.* To illustrate this method the data of Illustrative Prob. 51 (*a*) will be used. Fig. 92 (*a*) shows the funicular polygon drawn for the load and span, in which the half-span (25′-0″)

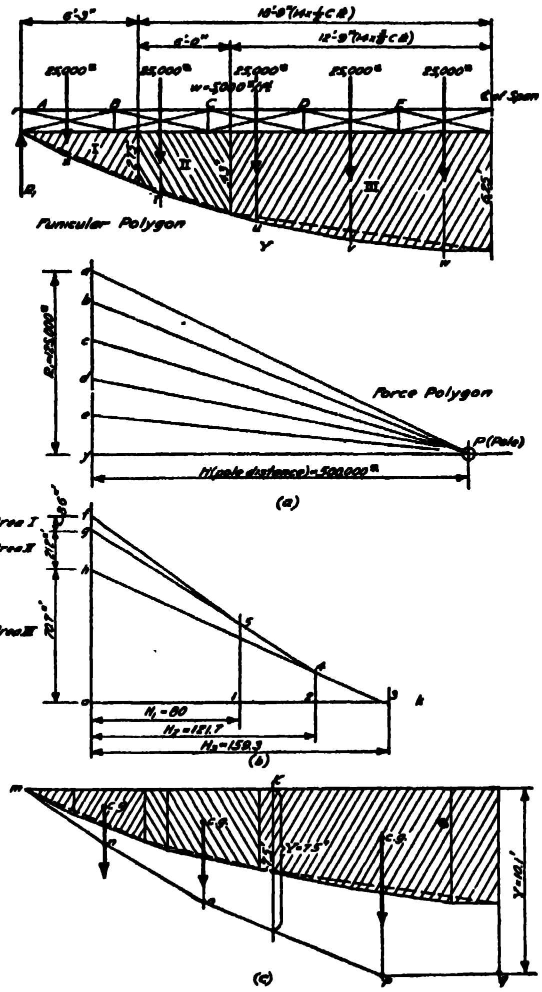


Fig. 92

is divided into 5 equal spaces (5′-0″ each). A curve tangent to the funicular polygon would represent the moment curve. The following steps are now taken:

(1) Divide the diagram *rstuvwx* into parts corresponding to the changes in flange section, as I, II and III. The moments of inertia between these intervals are then constant.

* For a more extended discussion of this method, see C. W. Malcolm's "Graphic Statics," M. C. Clark Publishing Co.

(2) Compute the area of each part in square feet, using the scale to which the beam was drawn. (Use average dimensions.)

$$\text{Area I} = \frac{6.25 \times 2.75}{2} = 8.6\square'$$

$$\text{Area II} = \frac{2.75 + 4.30}{2} \times 6.0 = 21.2\square'$$

$$\text{Area III} = \begin{cases} \dfrac{4.3 + 6.25}{2} \times 12.75 = 67.3\square' & \text{(trapezoid)} \\ \dfrac{13.9 \times 0.5}{2} = 3.4 & \text{(triangle)} \end{cases}$$

$$\underline{70.7\square'} \text{ approximately.}$$

(3) Calculate the moment of inertia of the girder in each part.

From Illustrative Prob. 51*a*,

I (for part I) = 23,075″⁴

I (for part II) = 36,075″⁴

I (for part III) = 46,110″⁴

(4) Determine the ratio of I at each section to the minimum I.

For area I, ratio = 1.00

$$\text{For area II, ratio} = \frac{36{,}075}{23{,}075} = 1.52$$

$$\text{For area III, ratio} = \frac{46{,}110}{23{,}075} = 1.99.$$

(5) Lay off the moment areas to a convenient scale as in Fig. 92 (*b*), on the vertical line *fgho*.

(6) Construct a horizontal line *ok* from *o*. Lay off H_1 upon this line equal to any value (say, 80□′).

(7) Since the deflection varies inversely as the moment of inertia of the section at the point under consideration, lay off

$$H_2 = H_1 \times \frac{I_2}{I_1}, \quad \text{and} \quad H_3 = H_1 \times \frac{I_3}{I_1}$$

$\therefore\ H_2 = 80 \times 1.52 = 121.7,$

and $H_3 = 80 \times 1.99 = 159.3.$

These values locate the points 2 and 3.

(8) Join points *h* and 3. Then draw 2-4, *g*-4, 1-5 and *f*-5 in their respective order and position, as shown.

(9) Locate the centers of gravity of the areas I, II and III (Fig. 92 (*a*)).

c.g. of area I $= 6.25 \times \frac{2}{3} = 4.17'$ from R_1

$$\text{Area II}, x = \frac{d\,(B + 2\,b)}{3\,(B + b)} = \frac{6\,(4.3 + 2 \times 2.75)}{3\,(4.3 + 2.75)} = 2.78'$$

c.g. $= 6.25 + 6.00 - 2.78 = 9.47'$ from R_1

$$\text{Area III}, x = \frac{12.75\,(6.25 + 2 \times 4.3)}{3\,(6.25 + 4.3)} = 6.0'$$

c.g. $= 25.0 - 6.0 = 19.0'$ from R_1

Figure 92 (*c*) shows these located.

(10) With *f*-5, *g*-4 and *h*-3 as rays, draw the funicular polygon *mnopq*. This represents the deflection diagram. Scale the distance Y (= 10.1′).

$$D = \frac{1728\, H \cdot H_1}{E \cdot I_{\min}} \times \text{intercept } Y^* \qquad (S\text{–}24)$$

$$D = \frac{1728 \times 500{,}000 \times 80}{30{,}000{,}000 \times 23{,}075} \times 10.1 = 1.01''.$$

The deflection at any other point may be calculated by proportion, once the deflection diagram is established. Thus

$$\frac{Y}{Y_1} = \frac{D}{D_1}, \quad \text{or} \quad D_1 = \frac{D \cdot Y_1}{Y}.$$

Thus the deflection at the point K in Fig. 92 (c) is

$$D_1 = \frac{1.01 \times 7.5}{10.1} = 0.75''.$$

Prob. 51b. Determine the approximate value of the maximum deflection for a plate girder carrying a load of 3600#/ft. on a 62'-0" span. Web plate 74 × 3/8, flange angles 6 × 6 × 7/8, one cover plate 14 × 1/2 × 49'-0", one cover plate 14 × 1/2 × 39'-0", one cover plate 14 × 1/2 × 28'-0".

Section 5D

DETERMINATION OF COVER PLATE LENGTHS

52. Practical Considerations.

The next step in the design of a girder is that of finding the length of the cover plates. The flange angles naturally run the full length of the girder, but the cover plates may be cut off at points where the additional area supplied by them is no longer required for moment resistance. Some designers specify that the cover plate adjacent to the flange angles should run the full length of the girder. This may be done for exposed girders, crane runways, and in railroad work to save the dapping of timbers. The plate also helps to stiffen the girder laterally. However, in building work one cover plate does not necessarily have to run to the ends, as the girder is generally covered by fire protecting materials in some manner.

The actual lengths of cover plates should be **1'-0"± greater at each end than** the calculated **theoretical lengths.** This allowance covers any slight errors in the determination of the theoretical length, and provides a space to partially develop the strength of the plate by rivets before the plate is theoretically required. Lengths of cover plates should be to the **nearest foot.**

53. Methods Used.

The theoretical lengths of cover plates may be determined analytically or graphically, for either uniform or combined loads.

The bending moment curve for a **uniform load** is a parabola. A property of the parabola is that the ordinates from a fixed horizontal line vary as the squares of the abscissæ from a fixed vertical line through the vertex. The maximum moment is proportional to the flange area, and likewise the moment at any point is proportional to the flange area required at that point. Hence the ordinates may be called the flange areas required. The property of the parabola may then be used to find **analytically** the distance from the center line of the span to the point where the flange area with a cover plate deducted will be sufficient to provide an ample resisting moment. In Fig. 93,

L = the span of the girder in feet,
A_{FN} = the total net area of the flange,
A_{CN} = the net area of all the plates up to and including the plate desired to be cut off, beginning with the outside plate, and
x = the distance in feet from the center line of span to the point under consideration.

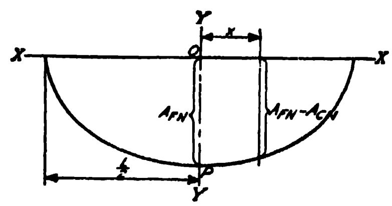


Fig. 93

Then

$$\frac{A_{FN}}{A_{FN} - A_{CN}} = \frac{\left(\frac{L}{2}\right)^2}{x^2}, \text{ or } x^2 = \frac{(A_{FN} - A_{CN})\left(\frac{L}{2}\right)^2}{A_{FN}},$$

and
$$x = \frac{L}{2}\sqrt{\frac{A_{FN} - A_{CN}}{A_{FN}}}.$$

But x is only one-half the theoretical length of the cover plate.

Let L_c = the length of the cover plate in feet $= 2\,x$.

Then
$$L_c = L\sqrt{\frac{A_{FN} - A_{CN}}{A_{FN}}}. \qquad (S\text{–}25)$$

To the values of L_c should be added 1'-0"± for each end and the length expressed to the nearest foot.

* For a more extended discussion of this method, see C. W. Malcolm's "Graphic Statics," M. C. Clark Publishing Co.

Illustrative Prob. 53a. Calculate the length of cover plates required for the following section uniformly loaded:

$42 \times \frac{5}{16}$ Web Plate. 40′-0″ span.
$2\text{-}6 \times 6 \times \frac{3}{8}$ Flange Angles
$2\text{-}14 \times \frac{3}{8}$ Cover Plates
Net area of 2 ∟s (2 holes out of each) = 8.80
Net area of 2 Pls. (2 holes out of each) = 9.18
$\frac{1}{8} A_W$ = 1.64
A_{FN} = 19.62□″

$$L_c = L\sqrt{\frac{A_{FN}-A_{CN}}{A_{FN}}} = \sqrt{\frac{9.18}{19.62}} \times 40 = 27.4$$

Use 1 Pl. 29′-0″ long.

$$L_c = \sqrt{\frac{4.59}{19.62}} \times 40 = 19.4$$

Use 1 Pl. 21′-0″ long.

Graphically, one-half of the span may be represented to a convenient scale, as in Fig. 94. At the center line of the span, B, an ordinate representing the maximum moment $\left(= \frac{w \cdot L^2}{8}\right)$ is then plotted to a fairly large scale, as the distance BC. The moment curve will be a parabola passing through the points A and C. Instead of calculating moments

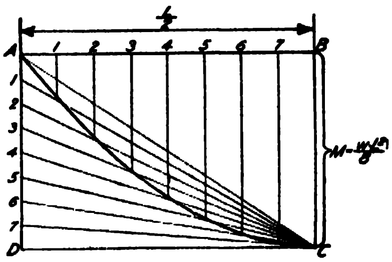


FIG. 94

at various other points and plotting the values corresponding, the parabola can be drawn when the points A and C are established, by the following construction:

> Complete the rectangle on the sides AB and BC, as $ABCD$. Divide AB into any number of convenient parts as A–1–2–3–4–5–6–7–B. The more parts chosen the more accurately the curve may be plotted. Divide the distance AD into the same number of parts as A–1–2–3–4–5–6–7–D. Connect C with points A, 1, 2, etc. Drop verticals from the horizontal line AB at points 1, 2, 3, etc., to intersect the diagonals corresponding with the same numbers. These intersections determine points on the parabola.

Since the total net flange area, A_{FN}, corresponds with the maximum moment, the next step is to superimpose this value (including $\frac{1}{8} A_W$, if used) to scale on the line BC, Fig. 95. To make two random scales coincide would be difficult. A simple way is to draw a line BR at any angle and plot the areas making up the flange upon it. These values should be plotted in order, starting with $\frac{1}{8} A_W$ (if used), BS, the net area of the flange angles, ST, then the cover plates in order, TU, UV and VR. Join C and R. Through V, U, T and S, draw lines parallel to CR, cutting BC. These lines will inter-

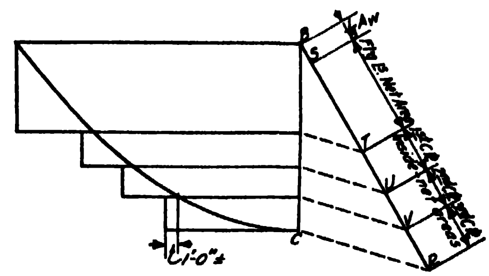


FIG. 95

cept the corresponding values of the component parts of the flange area at a scale coincident with that to which the maximum moment was plotted. Where these inclined lines intersect the vertical BC, draw the horizontal lines, as shown, to intersect the moment curve. These points of intersection determine the theoretical points where the cover plates may be discontinued. These distances are scaled off the diagram and 1′-0″± is added as explained above.

Illustrative Prob. 53b. Refer to Art. 39 and Fig. 71, for a graphical solution for uniform loading.

In the **graphical solution for any condition of loading,** the moment diagram must be laid out to scale, as in any case. The line representing the variation in moment across the span in this case may not be a simple parabola, but will be a combination of parabolas and straight lines. Moments should be calculated at all the critical points along the span (under all concentrated loads and where the rate of uniform load changes), and at enough other intermediate points to establish the curve correctly.* The curve should also be determined for the whole span if the loads are not symmetrical about the center line. The total net flange area is superimposed on this diagram at the point of maximum moment in the same manner as illustrated for the uniform load. The cover plate cut-off points can then be determined as before. For loads unsymmetrically arranged, the points of cut-off would be different on each side of the point of maximum moment, but for small differences it would probably be wiser to keep the lengths of cover

* The tenth points of the span are often used as critical points.

plates symmetrical about the center line of the girder, as governed by the worse condition. Advantages would be gained in detailing, use of templates, and fabrication, by this uniformity. There are cases, however, where this is not feasible. The designer must exercise his judgment in this respect.

Illustrative Prob. 53c. Determine the lengths of cover plates graphically for the loading shown in Fig. 96 for the following section:

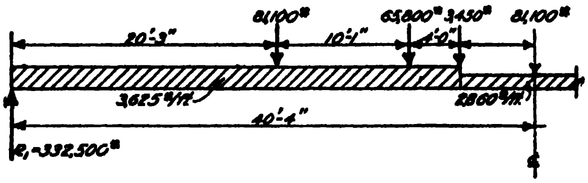


FIG. 96

72 × ½ web plate	⅛ A_W =	4.50□″
2 ∟s 8 × 8 × ⅞ (3 holes out each angle)	=	21.18□″
2-12 × ⅝ flange plates (2 holes out of each)	=	15.00□″
3-20 × ⅞ cover plates (2 holes out of each plate) for each plate.	=	15.75□″

Figure 97 shows the graphical solution. Compare with Table 27. Refer to Art. 76 and Fig. 130 for another typical solution.

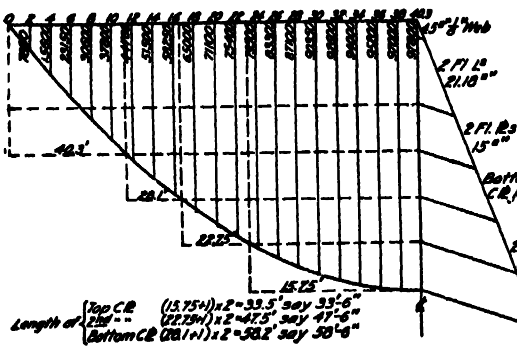


FIG. 97

No definite formula can be used for an **analytical solution for varied loading.** The solution involves a quadratic equation. The resisting moment of the flange section without any cover plate, using the proper effective depth, should first be calculated. This value is then equated to the general expression for the bending moment at any point, x distant from the support. $\left(M_x = R_1 \cdot x - \frac{w \cdot x^2}{2}\right)$. The value of x as determined will be the theoretical distance to the point where the first cover plate should start. If the value of x were greater than the distance to the first concentrated load, the general equation for the moment would have to be adjusted and solved again. In a similar manner the length of the second cover plate could be determined by equating the resisting moment of the net flange, including the first cover plate, to the external moment, x distant from the support, taking care to deduct the moments of any concentrated loads.

Illustrative Prob. 53d. Check the length of the inside cover plate in Illustrative Prob. 53c (Fig. 96).

$$M_x = R_1 \cdot x - \frac{w \cdot x^2}{2} = 332{,}500 \cdot x - \frac{3625 \cdot x^2}{2}$$

Net section with no cover plates = 4.50 + 21.18 + 15.00 = 40.68□″. d_e = 67.2″.

$$M_r = A_{FN} \cdot d_e \cdot f_s = 40.68 \times 67.2 \times 16{,}000 = 44{,}700{,}000''\# = 3{,}729{,}000'\#$$

$$3{,}729{,}000 = 332{,}500 \cdot x - \frac{3625 \cdot x^2}{2} \qquad (M_r = M_x)$$

$$x^2 - 182\,x = 2060$$
$$x^2 - 182\,x + (91)^2 = 2060 + (91)^2 = 10{,}460$$
$$x - 91 = \pm\sqrt{10{,}460} = \pm 102.5$$
$$x = 11.5$$

40.3 − 11.5 = 28.8 $\qquad$ 2 × 28.8 + 2′-0″± = 59′-0″.
(Compare with Fig. 97.)

In a similar manner the computations may be carried out for the other plates.

The **above solution** is at times **varied to avoid quadratics.** In this method the first step is to calculate the moments along the girder at intervals sufficiently close to obtain the desired accuracy. These should be tabulated, as shown in Table 27, in thousands of foot-pounds. Then,

$$\frac{I}{c} = \frac{M \text{ in inch-lbs.}}{16{,}000},$$
$$\frac{I}{c} = \frac{M \text{ in ft.-lbs.} \times 12}{16{,}000},$$
$$\frac{I}{c} = \frac{\text{“}x\text{” thousands of ft.-lbs.} \times 1000 \times 12}{16{,}000}, \text{ or}$$
$$\frac{I}{c} = 0.75\,(\text{“}x\text{”}).$$

The second column can be made by multiplying the values in the first column by 0.75. Also,

$$A_{FN} = \frac{I}{c} \div d_e.$$

The values of d_e should be tabulated in the third column. Where there are no cover plates, the effective depth may be established from the known flange section. This value may be used until it is known that a cover plate becomes necessary. Then the new value of d_e may be used (with one cover plate) until the second cover plate is required, and so on. The fourth column may be established by dividing the values in the third column by d_e. When this is established part way, it may be compared with the net flange area without cover plates

TABLE 27

ANALYTICAL SOLUTION FOR COVER PLATE LENGTHS — COMBINED LOADING*

	1	2	3	4	5 (Actual Areas)
Pt.	1000′#	$\frac{I}{C}$″³	d_e″	A_{NC}□″	$A_{NC} = \frac{M''\#}{S \cdot d_e}$
M_2	657	493		7.0	
M_4	1301	975		13.9	
M_6	1930	1447		20.6	4.50 = 1/8 web
M_8	2544	1910	64.6	27.2	21.18 = 2 fl. ∟s
M_{10}	3144	2360		33.6	15.00 = 2 fl. Pls.
M_{12}	3729	2800		39.9	40.68
M_{14}	4300	3220		45.9	15.75 = 1 cover Pl.
M_{16}	4856	3640	67.2	51.8	56.43
M_{18}	5402	4050		57.7	
M_{20}	5925	4440	69.0	63.2	
M_{22}	6297	4720		67.3	15.75 = 1 cover Pl.
M_{24}	6632	4960		70.6	72.18
M_{26}	6943	5200		74.1	
M_{28}	7261	5450		77.6	
M_{30}	7532	5650		80.4	
M_{32}	7722	5780	70.4	82.3	
M_{34}	7841	5880		83.8	
M_{36}	7931	5950		84.7	
M_{38}	8083	6060		86.4	15.75 = 1 cover Pl.
$M_{40.3}$	8155	6110		87.5	87.93

72 × 1/2 web Pl. 2 ∟s 8 × 8 × 7/8 (3 holes out each angle) = 21.18□″.

2-12 × 3/4 flange plates (2 holes out of each) = 15.00□″.

3-20 × 7/8 cover plates (2 holes out of each plate) = 15.75□″ for each plate. Span 80.7′.

Top cover Pl. can be cut off between the 24′ and 26′ points, interpolate for exact length 24.8+′.

2nd cover Pl. cut off between 16′ and 18′ or 17.6′ exact.

Bottom cover Pl. cut off between 12′ and 14′ or 12.25′ exact.

Length of:
- top cover Pl. 80.7−(2×24.8)+2=33.1′ say 33′-0″.
- 2nd " " 80.7−(2×17.6)+2=47.5′ say 47′-6″.
- bottom " " 80.7−(2×12.25)+2=58.2′ say 58′-6″.

*For Loading diagram, see Fig. 96.

to note where the value in this column exceeds the value of the available flange area. It is evident that to supply area enough beyond this point it is necessary to add a cover plate. This procedure may be carried on for the other cover plates. It will be close enough to interpolate the length between values of the moment tabulated, as shown in the table.

The selection of the **method to use** depends upon the engineering judgment and the preference of the designer. For uniform loads only, probably the analytical solution is more direct. For simple conditions of loading, such as a concentrated load at the center of a span, concentrated loads at the third points and the like, the graphical method is short because the moment curve is readily established. The last analytical solution given has advantages in that no knowledge is required of the method of solving a quadratic equation. It is also more direct where there are few concentrated loads.

Prob. 53e. Calculate the required length of the cover plate for the following conditions:

M_{max} = 8,000,000′# Uniform load.
48 × 3/8 web plate (1/8 A_W available for flange material)
6 × 4 × 5/8 flange angles (2 holes out each angle for 3/4″ rivets)
1-14 × 3/8 cover plate (2 holes out)
Span 40′-0″.

Prob. 53f. Determine the required length of the cover plate in Prob. 53*e* by graphical method.

Prob. 53g. Check the lengths of cover plates determined in Art. 39 by an analytical solution.

Prob. 53h. From the data in Art. 76, verify the results of the cover plate lengths by laying out your own moment diagram and obtaining the lengths graphically.

Section 5e

STIFFENERS

54. The Purpose of Stiffeners.

When stiffeners are used, they form natural panel points at which the pitches of rivets required to connect the flange angles to the web plate may be calculated. This is an obvious reason why the location of the stiffeners is the next logical step in plate girder design.

Stiffeners function in the following ways:

(1) they hold the web plate true to shape during the fabrication and erection,

(2) they prevent the web from buckling,

(3) they add torsional rigidity,

(4) they aid in resisting the compression developed by lateral forces in the top flange,

(5) they relieve the rivets in the top flange when it is loaded directly, and

(6) they reduce the web stress where concentrated loads occur.

The design of stiffeners is divided into two main groups. There are those which are at concentrated loads (also at end reactions) and those which are placed between these points to keep the web from buckling. Those placed between the stiffeners at concentrated loads are called intermediate stiffeners. Stiffeners at concentrated loads or reactions obviously resist direct compression, like columns would, while the intermediate stiffeners act more indirectly, but nevertheless are similar to small columns. The primary purpose of stiffeners is then to assist the web plate in carrying the shear so that buckling may not occur. They are often a convenience for beam connections.

55. Stiffeners at Concentrated Loads.

There are two methods commonly used by designers for determining the sizes of angles required as stiffeners at concentrated loads. One method involves the assumption that the load is taken by

the angles acting as a column; the other assumes that the load is taken by the metal which is in direct bearing only.

In the former of these methods there is a difference in practice as to what the effective cross-section should be. One assumption is that the **metal** held **between the stiffener angles is a part of the section.** Thus, as in Fig. 98 (*a*), the angles, fillers* and the strip of the web covered by the stiffeners may be considered as the cross-section. Since the stiffener is more or less restrained from sidewise bending by the rivets, the effective length of the "column," *l*, is assumed to be one-half of the depth of the girder, as in Fig. 98 (*b*). This is conservative, as

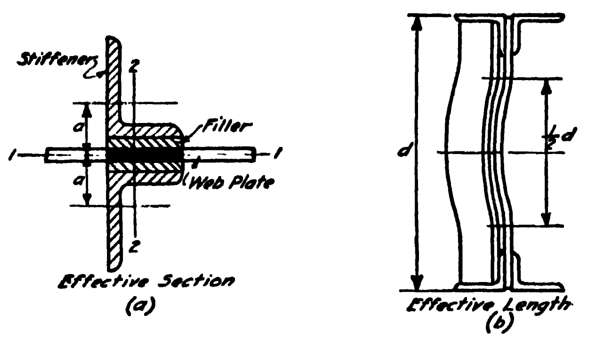


Fig. 98

in many cases the stiffener receives and discharges its load in increments. The only radius of gyration which need be considered is that at right angles to the web, or about axis 1–1 in Fig. 98 (*a*), because the continuity of the web plate eliminates the tendency of bending about the 2–2 axis. Any accepted column formula may be used, and the following one is suggested:

$$p = 16{,}000 - 70\frac{l}{r}.$$

$$\text{Since } l = \frac{d}{2}, \qquad p = 16{,}000 - \frac{35\,d}{r}. \qquad (S\text{–}26)$$

Illustrative Prob. 55a. Design a pair of stiffener angles to take a load of 65,800#. 8 × 8 flange angles and 72 × ½ web plate. Use column method ($l = \frac{d}{2}$).

For 8 × 8 flange $\llcorner^s$ use 6 × 3½ stiffener angles (Table 28). Try minimum size, namely 6 × 3½ × ⅜.

$$A = 3.42\square'', \qquad I_{1\text{-}1} = 12.9''^4.$$

Distance "*a*" in Fig. 98 (*a*) = 2.04 + ⅞ + ¼ = 3.16″

$$I_0 = I + A \cdot r^2 = (12.9 + 3.42 \times \overline{3.16}^2) \times 2 = 94.2''^4.$$

The moment of inertia of the fillers and the portion of the web plate about their own axes will be neglected.

$$A = 2 \times 3.42 + 2 \times 3\tfrac{1}{2}\,(2 \times \tfrac{7}{8} + \tfrac{1}{2}) = 22.59\square''$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{94.2}{22.6}} = 2.04''$$

$$d = 72'' \text{ approximately, or } l = 36''$$

$$p = 16{,}000 - \frac{70 \times 36}{2.04} = 13{,}530\#/\square''$$

14,000#/□″ is often specified as the maximum allowable for the above formula.

$$P \text{ (safe load)} = 13{,}530 \times 22.59 = 306{,}000\#$$
$$p \text{ (actual)} = 65{,}800\#$$

Hence 2-6 × 3½ × ⅜ angles are amply safe. The above computations should illustrate the unnecessary calculations for stiffener design if proper rules are followed.

When the alternate assumption, that the **concentrated load is carried as a direct stress** by the area only, is made, a fixed stress (13,000#/□″ is common) may be employed and the formula

$$A = \frac{P}{p}$$

used.† It is wise to calculate the area required by both methods and use the larger. In the usual case the theoretical area of the angles required will be

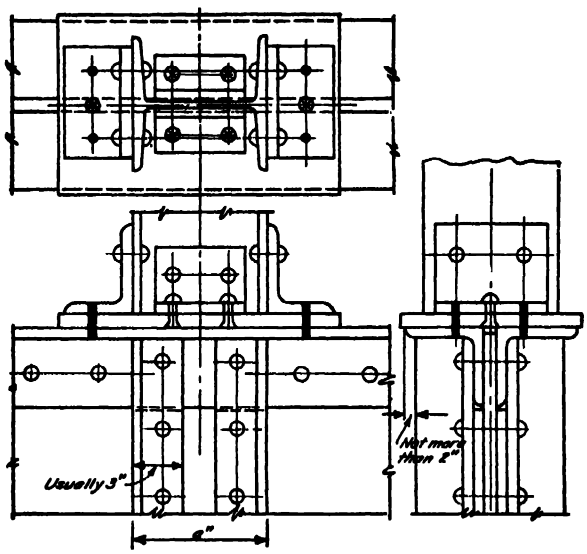


Fig. 99

much smaller than that which must be used for practical reasons. The angles should always provide a direct bearing for the section of a column which is superimposed upon the girder flange and should also have long enough outstanding legs to give the flange angles support (Fig. 99). The leg which is riveted to the web should be at least 3″. The use of $l = d$ is often substituted for the values used here, but in either case the above practical considerations control.

* Some designers question the point of including the fillers as they are loose. However, they are riveted as much as the angles themselves are.

† The ratio of slenderness, $l \div r$ is quite small in such cases,—hence the use of a fixed stress.

Illustrative Prob. 55b. Design the stiffener angles for the data of Illustrative Prob. 55a by the direct stress method.

$$A = \frac{P}{p} = \frac{65{,}800}{13{,}000} = 5.06\square'' \text{ required.}$$

$$\text{2-6} \times 3\tfrac{1}{2} \times \tfrac{3}{8} \text{ Ls} = 2 \times 3.42 = 6.84\square'' \text{ actual. O.K.}$$

The area in the angles only is usually sufficient without including the portion of the metal included between them.

The more **common method** of design is to make certain that the stiffener angles have sufficient area in the outstanding legs so that the allowable **bearing stress is not to be exceeded.** It is impracticable to secure accurate bearing against the fillets of the flange angles, so that the stiffeners are generally beveled off at a 45° angle to prevent them from fouling the fillets, as shown in Fig. 100. The "net" bearing length is then the width of the outstanding leg of the stiffener angle minus the radius of the fillet of the flange angle. The allowable bearing stress in this case is usually taken as 20,000#/□″. The end of the stiffener is milled to secure accurate bearing. In fabrication the attempt to make both ends bear accurately often fails, and in detailing, the bearing end is often marked B.E. to make certain that the detail is not overlooked.

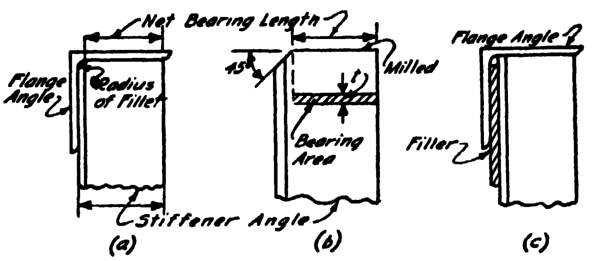


Fig. 100

Illustrative Prob. 55c. Design the stiffener angles for the data of Illustrative Prob. 55*a* by the direct bearing method.

$$\text{Radius of fillet of an } 8 \times 8 \text{ L} = \tfrac{5}{8}''$$
$$\text{Net bearing length} = 6 - \tfrac{5}{8} = 5\tfrac{3}{8}'' = 5.37$$
$$\text{Bearing area required} = \frac{65{,}800}{20{,}000} = 3.29\square''$$
$$\text{Thickness of angle} = \frac{3.29}{2 \times 5.37} = 0.308''$$

Use 2 Ls $6 \times 3\tfrac{1}{2} \times \tfrac{3}{8}$ for practical reasons.

It should be obvious by comparing the three results in Illustrative Probs. 55*a*, 55*b* and 55*c* that the same size of stiffener angles is obtained in each case. This is usual. However, the last method is the most conservative and the nearest to actual bearing conditions, and it is recommended by the authors.

Some designers prefer to introduce a filler of a thickness equal to the radius of the fillet, thereby developing the full bearing area of the stiffener angle, as in Fig. 100 (*c*). While additional area is obtained in the bearing of the stiffener angle, the extra cost of the filler will usually more than offset this advantage, as well as the cost of grinding the corners. Due to the questionable bearing efficiency of such a combination, it is deemed wise to reduce the allowable bearing stress to 16,000#/□″ in this case.

The outstanding legs should extend as far as the flange angles will allow in order to properly brace the flange. The width of the legs adjacent to the web plate should be wide enough to allow the rivets to be driven. This is usually 3″. The angles should be placed on fillers so that crimping may be avoided, as shown in Fig. 102 (*c*). Some of the common sizes of stiffener angles used are shown below.

TABLE 28

COMMON SIZES OF STIFFENER ANGLES

Flange Angles		Stiffeners	
Horiz. Leg	Thickness	At Conc. Loads	Intermediate
4″	Any	$3 \times 3 \times \frac{1}{2}$	$3 \times 3 \times \frac{3}{8}$
5	Any	$4 \times 3 \times \frac{1}{2}$	$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$
6	$\frac{3}{4}$″ and under	$5 \times 3\frac{1}{2} \times \frac{1}{2}$	$4 \times 3 \times \frac{5}{16}$; $5 \times 3\frac{1}{2} \times \frac{7}{16}$
6	Over $\frac{3}{4}$″	$5 \times 3\frac{1}{2} \times \frac{5}{8}$	$5 \times 3\frac{1}{2} \times \frac{1}{2}$
8	Any	$6 \times 6 \times \frac{1}{2}$; $6 \times 6 \times \frac{3}{4}$	$6 \times 3\frac{1}{2} \times \frac{3}{8}$; $6 \times 6 \times \frac{5}{8}$

Prob. 55d. Design a pair of stiffener angles to carry a concentrated load of 80,000#. 6 × 6 flange angles and a 60 × $\frac{3}{8}$ web plate. Design by the three methods and compare results.

Prob. 55e. Design the stiffener angles to carry a column load of 200,000# similar to that shown in Fig. 99. Use direct bearing method.

56. Stiffeners at End Bearings.

As has been stated, the purpose of these stiffeners is to receive the shear from the web and to transmit it to the bearing plates. The action involved is similar to that of the buckling at the bearing of a rolled section (Art. 13). Fig. 101 (*a*) shows a common method of placing a pair of stiffeners at the outer and inner edges of the bearing plates and in between these points if necessary. An objection to this method is that the inner stiffeners in reality take more than their share of the load. A better method is to place the stiffeners over the center of bearing as shown in Fig. 101 (*b*), and to design the bearing plate for concentrated loading. The first method, however, gives a little more finish to the girder and is convenient when the girder is to be connected to the faces of columns. The procedure in obtaining the size of angles required

is the same as that given for other concentrated loads (Art. 55).

Illustrative Prob. 56a. Design the stiffener angles in Fig. 101 (*a*) if the end reaction is 125,000#. Web plate $60 \times \frac{3}{8}$, flange angles $6 \times 4 \times \frac{1}{2}$. Use direct bearing method.

Use $5 \times 3\frac{1}{2}$ ∟s for 6×6 flange angles.
Radius of fillet of a 6×6 angle $= \frac{1}{2}''$
Net bearing length $= 5 - \frac{1}{2} = 4\frac{1}{2}''$

$$\text{Bearing area required} = \frac{125{,}000}{20{,}000} = 6.25\square''$$

$$\text{Thickness required} = \frac{6.25}{4.5} = 1.39''$$

$$\frac{1.39}{4} = 0.35'' \qquad \text{Use 4-}5 \times 3\tfrac{1}{2} \times \tfrac{3}{8} \text{ ∟s.}$$

Prob. 56b. Design the stiffeners for an end reaction of 332,000#. Web plate $72 \times \frac{1}{2}$, flange angles $8 \times 8 \times \frac{7}{8}$.

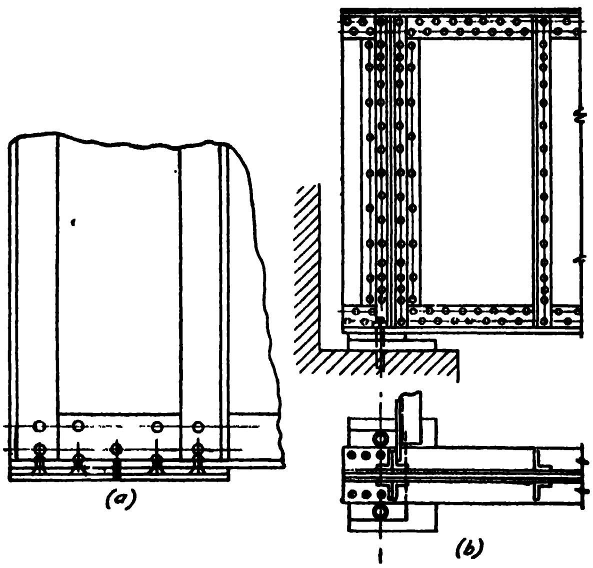


FIG. 101

57. Types of Intermediate Stiffeners.

The relation of the stiffeners to the web plate may be fixed in two ways: one in which the angles are placed on fillers so that they may run directly on to the flange angles as in Fig. 102 (*a*), and the other in which the angles are crimped to offset them enough to produce the same effect, as in Fig. 102 (*b*) without the use of fillers. The first method is the more commonly used, as small fabricators are not always equipped to do crimping and the cost of such work usually exceeds that of the fillers, unless there is considerable duplication. In Europe the relation of costs of material and labor are different than in the United States and crimping is used more. Crimped angles are slightly more efficient in restraining the web, but are not as trim and require more careful workmanship. They should never be used where direct stress is to be transmitted, such as at concentrated loads and end reactions, or where holes must be used in the outstanding legs. Intermediate stiffeners need not be milled to bear. If a girder is to be encased in concrete, the stiffeners may be cut off at the edge of the fillet of the flange angle. A third method occasionally used is shown in Fig. 102 (*c*). It is used principally in bridge work but occasionally in building girders to brace wide cover plates, or to offer additional lateral rigidity. This type of knee brace is good design, but it is more expensive and usually not necessary in building work.

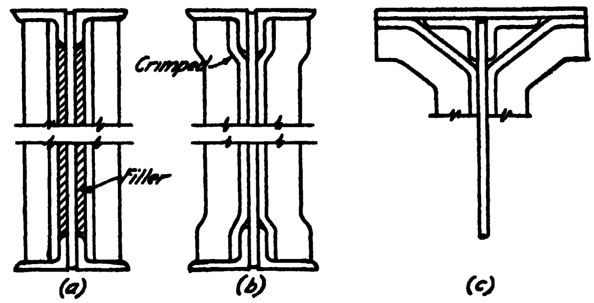


FIG. 102

58. Practical Requirements for Intermediate Stiffeners.

The stress in an intermediate stiffener is indeterminate and the arrangement must be left somewhat to good judgment and experience, and has been provided for by several empirical specifications:

SPECIFICATION CLAUSES

When Required If the thickness of web plate is less than one-sixtieth of the unsupported distance between the flange angles, intermediate stiffeners shall be used.*

Outstanding Legs The width of the outstanding legs of the intermediate stiffener angles shall not be less than 2″ plus one-thirtieth the depth of the web plate.

Expressed as a formula, $w_s = 2'' + \frac{h}{30}$.† (S–27)

* Another basis of requirement is that of comparing the maximum intensity of shear with an allowable which is based upon the Gordon formula and is as follows:

$$v = \frac{15{,}000}{1 + \frac{d^2}{3000\,t^2}}, \text{ in which}$$

d = the unsupported distance between the flange angles, and
t = the thickness of the web plate, both in inches.

If the actual shear is less than this allowable, no stiffeners are required.

Another basis of determining whether intermediate stiffeners are required or not, is by the use of a formula given in M. S. Ketchum's "Structural Engineers' Handbook" — Copyright, McGraw-Hill Publishing Co., which is

$v = 12{,}500 - 90\,H$, in which
H = the ratio of the depth of the web to its thickness, and
v = the allowable stress on the gross section.

If the actual web stress is less than v, no intermediate stiffeners are required.

† Some specifications give 32 for the constant instead of 30.

above are common guides to determine the details for intermediate stiffeners. In some cases, however, it is more economical to increase the thickness of the web plate sufficiently to eliminate intermediate stiffeners. Stiffeners obtain their greatest strength by having the longer legs outstanding and extending nearly to the toes of the flange angles. In this manner they also offer a better resistance to any lateral deflection of the top flange. The nearest stock-size angle, which meets the above requirement, must be used. The width of the leg adjacent to the web is determined as before (Art. 55), and is usually 3″ (Table 28).

Illustrative Prob. 58a. What size of intermediate stiffener angles should be used for a girder composed of a 60 × 3/8 web plate and 4-6 × 6 × 3/4 flange angles?

$$\frac{h}{30} + 2 = \frac{60}{30} + 2 \quad 4'' \qquad \text{Use } 4 \times 3 \times \tfrac{5}{16} \text{ Ls.}$$

Prob. 58b. What size of intermediate stiffener angles should be used in a plate girder composed of a 72 × 1/2 web plate and 8 × 8 × 7/8 flange angles?

59. Spacing of Intermediate Stiffeners.

Intermediate stiffeners are used to provide for the tension and the compression which exist at right angles to each other at any point in the cross-section of a plate girder web. The inclinations of these stresses with respect to the horizontal depend upon the relative values of the shear and the bending stress. Where the external shear is a maximum and the external moment a minimum, the inclinations are practically at 45° with the horizontal. Where the external shear is 0, and the external moment is a maximum, the bending stress alone exists and the stress is therefore parallel to the neutral plane. It may be shown that the inclinations of these resultant stresses vary from one extreme condition to the other.*

The existence of the inclined compression has a very important influence on the web, having a tendency to cause it to buckle in a sidewise direction. The critical point is where the shear is a maximum, and here the web should be restrained more than at points where the vertical shear has decreased. The web must be made thick enough to resist this tendency to buckle, or it must be restrained by the introduction of intermediate stiffeners. Such stiffeners should occur in pairs, as one placed on one side of the web would not be entirely effective in preventing the web from buckling and also would tend to put the connecting rivets in direct tension.

There is **no exact method** of analyzing the functioning of intermediate stiffeners. Experiments have been few and inconclusive, but experience has shown that the methods used in modern practice are safe. One of the methods, which has a theoretical basis,† is suggested here.

Let Fig. 103 (a) represent a panel of a plate girder where the shear is nearly maximum and consequently the moment very small. The inclined compression will be approximately at a 45° angle with the horizontal. Let d_s be the clear distance between stiffeners in inches. Consider a strip 1″ wide, the thickness of which is t.

Let I = its moment of inertia (ins.)^4,
r = its radius of gyration (ins.), and
A = the area in cross section of the strip (ins.)^2.

The column formula which is used as a basis is:

$$\frac{P}{A} = 16{,}000 - K \cdot \frac{l}{r}. \tag{1}$$

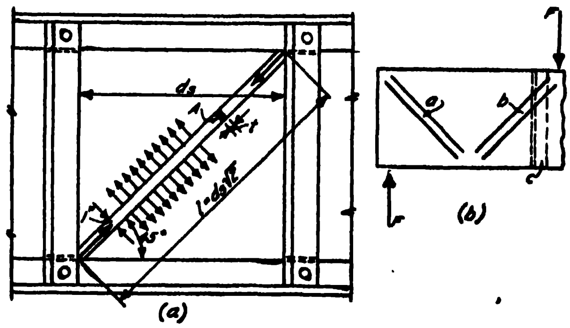


FIG. 103

Since this strip is restrained by the continuity of the web plate and there co-exists a web tension at right angles to the compression (as indicated by the arrows), which would also restrain the tendency toward sidewise buckling, the strip would not act like a simple column.‡ The constant K is empirically reduced from its usual value of 70 to 25† for these reasons, or

$$\frac{P}{A} = 16{,}000 - 25\frac{l}{r}. \tag{2}$$

The length of the "column" is restricted by the flange angles, and the effective length, l, will be considered to be

$$l \quad d_s\sqrt{2}. \tag{3}$$

But
$$\sqrt{\frac{I}{A}} = \sqrt{\frac{1 \times t^3}{12} \div (t \times 1)} = \frac{t}{\sqrt{12}}. \tag{4}$$

Substituting,

$$\frac{d_s\sqrt{2}}{\frac{t}{\sqrt{12}}} \quad \frac{d_s\sqrt{24}}{t}. \tag{5}$$

The limit for $\frac{l}{r}$ is set at 300.† This may seem exceptionally high, as $\frac{l}{r}$ is usually limited to 120 for main members and sometimes to 200 for secondary members in building work.

* Refer to any standard text on Elementary Mechanics.

† See "Theory of Structures," 1st Edition, by C. M. Spofford — McGraw-Hill Publishing Co.

‡ The functioning of an intermediate stiffener can be illustrated by making a cardboard model as shown in Fig. 103 (b), in which slots are cut as shown by the diagonal lines. If a couple is exerted on the model as indicated by the arrows "F," strip a will tend to tighten and b will tend to buckle. A stiffener superimposed at c would eliminate the buckling.

The strip of web, however, does not even act as a secondary compression member. The original formula then becomes:

$$\frac{P}{A} = 16{,}000 - 25 \times 300 = 8500\#/\square''.$$

This value of $\frac{P}{A}$ approaches the value allowed for web shear on the net section. As

$$\frac{d_s\sqrt{24}}{t} = 300,$$

then $$t = \frac{d_s}{60} \text{ (practically)}$$

This limit is the source of the specification commonly used (Art. 58). Substituting for K and using the value of $\frac{l}{r}$ given in (5) above,

$$\frac{P}{A} = 16{,}000 - 25\frac{d_s\sqrt{24}}{t},$$

and $$\frac{P}{A} = 16{,}000 - 120\frac{d_s}{t} \text{ (practically)}.$$

But $\frac{P}{A}$, the intensity of compression, is equal to the intensity of vertical shear, v. Hence

$$v = 16{,}000 - 120\frac{d_s}{t} = \frac{V}{A_{WN}}.$$

Solving for d_s, the spacing in the clear between stiffeners in inches for a given size of web at any point results, namely

$$d_s = \frac{t}{120}\left(16{,}000 - \frac{V}{A_{WN}}\right). \qquad (S\text{–}28)$$

The value of d_s is usually assumed as equal to the distance from the gauge line of one stiffener to that of the next. This formula gives conservative values There are also other formulas which are used, derived in a similar manner. One is

$$d_s = \frac{t}{40}\left(12{,}000 - \frac{V}{A_{WN}}\right).^* \qquad (S\text{–}29)$$

Some designers use the specification relative to one-sixtieth the unsupported distance between flange angles and vary the spacing according to

$$\frac{1}{60} \times \frac{S_a}{S_s},$$

in which S_a = the allowable intensity of shear,
and S_s = the actual intensity of shear.

This is again varied by using $d_s = 64\,t\left(\frac{16{,}000}{S_s}\right)$ in which S_s = the actual unit shear in the web at the point under consideration. The use of 64 here in place of 60, as before, is explained by the fact that some specifications state that stiffeners are required when the web thickness is less than $\frac{1}{64}$th of the depth between inner flange rivet lines. This relation has a definite connection with the spacing formula and the two values, 60 and 64, interchange similarly. By the above specifications the line of action of the compressive stress cannot cross the web plate at 45° without being intercepted by a stiffener. If the stiffeners were placed farther apart than the clear distance between the flange angles, the action of the strip of the web would be unaffected, and the stiffeners would not theoretically be of service.

* Corresponding to the Building Law of the City of Boston, and recommended for use.

SPECIFICATION CLAUSE

Spacing Limits Stiffeners shall not be spaced farther apart than the depth of the web plate nor in any case more than 5′-0″.

When the moment is large and the shear small, conditions do not approach the above theory, as the inclinations of the resultant stress lines are not at 45°, and the formulas apply only approximately. When d_s, as calculated, actually exceeds the specification just given, the stiffeners should nevertheless be spaced in accordance with it. Some designers simply use the rule without calculations spacing the stiffeners uniformly. It seems reasonable to make the spacing of stiffeners increase with an increment corresponding with the decrease in shear. The number of stiffeners required would be increased only slightly and they would be arranged on a more rational basis. The formula should be used for arranging stiffeners between the concentrated loads. It gives the required spacing at a given point so that the spacing would be more, or less, if calculated for a point to one side of that tried. Accordingly, judgment must be used in the arrangement of intermediate stiffeners. It is a good policy to make the spacing symmetrical about the center line of the span if possible. This simplifies the details and makes the stiffeners more interchangeable.

Illustrative Prob. 59a. Calculate the required spacings of the intermediate stiffeners at the left-hand end of the girder shown in Fig. 104.

Web plate $72 \times \frac{5}{8}$ Use $d_s = \frac{t}{40}\left(12{,}000 - \frac{V}{A_{WN}}\right)$.

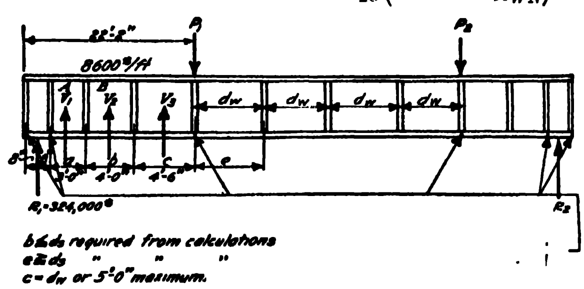


FIG. 104

C.L. of panel A assumed 8″ + 1′-4″ say 2′-0″ from R_1.

$$V_1 = 324{,}000 - 8600 \times 2 = 306{,}800\#$$

$$A_{WN} = \tfrac{3}{4} \times 72 \times \tfrac{5}{8} = 31.2\square''$$

$$d_s = \frac{0.625}{40}\left(12{,}000 - \frac{306{,}800}{31.2}\right) = 34.8'' \quad \text{Use } 3'\text{-}0''.$$

On the basis of the above calculation, assume C.L. of panel B, 8″ + 3′-0″ + 1′-10″ = 5′-6″ from R_1, as the next spacing certainly will be somewhat greater than the one previously calculated.

† For end stiffeners and those which occur at concentrated loads, see Arts. 55 and 56.

$$V_2 = 324{,}000 - 8600 \times 5.5 = 276{,}700\#$$

$$\cdot \frac{0.625}{40}\left(12{,}000 - \frac{276{,}700}{31.2}\right) = 47.8'' \quad \text{Use } 4'\text{-}0''.$$

Assume C.L. of panel C, $8'' + 3'\text{-}0'' + 4'\text{-}0'' + 2'\text{-}4'' = 10'\text{-}0''$ from R_1

$$V_3 = 324{,}000 - 8600 \times 10 = 238{,}000\#$$

$$d_3 = \frac{0.625}{40}\left(12{,}000 - \frac{238{,}000}{31.2}\right) = 68''$$

Use 5'-0" on account of specifications.

, From this point on, the spacing should be 5'-0", except to match stiffeners already at concentrated loads. Figure 104 shows a practical arrangement. It will be noted that the distance c is made 4'-6" (instead of 5'-0") in order to match the 22'-2" dimension locating the concentrated load P_1.

Prob. 59b. Determine an arrangement for a plate girder on 50'-0" span carrying a total uniform load of 5000#/ft. Plate girder section 60 × $\frac{5}{16}$ web and 6 × 4 × ½ flange angles. Assume inside pair of stiffeners at end bearing 6" in from center line of bearing.

Section 5F

CONNECTION OF THE FLANGES TO THE WEB

60. Design Methods.

As the fibres of a beam of any kind, when loaded, tend to shear along horizontal planes, the flange angles of a plate girder obviously are subject to sliding on the web plate. This action is offset by connecting rivets. The **exact solution** involves the calculation of the intensity of longitudinal shear, for which the formula

$$q = \frac{V \cdot Q}{b \cdot I}, \text{ or}$$

$$q \cdot b \quad \frac{V \cdot Q}{I} = \text{the shear per linear inch,}$$

is used. Since this stress increases from the extreme fibre to the neutral axis, the plane of maximum shear is at a–a, Fig. 105, the inside edges of the vertical legs of the flange angles, although the stress is transmitted through the rivets at b–b. The statical moment, Q, used in the formula then is that of the flange angles and cover plates, if any, about the neutral axis, but excluding the portion of the web between the flange angles, since the stress in this part is carried by the web itself. With Q and I established, the value of the longitudinal shear per linear inch $(q \cdot b)$ may be calculated. This is the force tending to slide the flange on the web, or the rate of the increase of the flange stress which must be carried by the rivets in the vertical legs of the flange angles. If a rivet is good for "R" pounds, and the longitudinal shear per linear inch is "x" pounds, then the pitch, or horizontal distance center to center of rivets, is

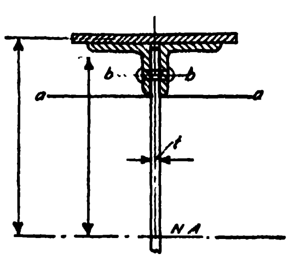


Fig. 105

$$p = \frac{R}{x}.$$

Illustrative Prob. 60a. Calculate the minimum pitch for a plate girder section of a 66 × $\frac{9}{16}$ web plate, 2-8 × 8 × ⅞ flange angles, 2-18 × ¾ cover plates and 1-18 × ⅝ cover plate, if the end reaction = 324,000#. Assume C.L. of first panel 2'-6" from R_1. Load 8600#/ft.

$$V_{2'\text{-}6''} = 324{,}000 - 8600 \times 2.5 = 302{,}500\#$$

No cover plates occur at this panel.
Statical moment of angles about neutral axis

$$Q = A \cdot d = 26.46 \times 30.93 = 820''^3$$

Moment of inertia of whole section about N. A.

$$66 \times \tfrac{9}{16}\text{ Pl.} = \frac{b \cdot d^3}{12} = \frac{0.5625 \times (66)^3}{12} = 13{,}470$$

$$I \text{ of } 4 \text{ ∟s} = 4 \times 79.6 = 320$$

$$A \cdot d^2 \text{ of } 4 \text{ ∟s} = 13.23 \times \overline{30.93}^2 \times 4 = 50{,}600$$

$$\text{Total} \quad 63{,}390''^4$$

$$b \cdot q = \frac{V \cdot Q}{I} = \frac{302{,}500 \times 820}{63{,}390} \quad 3950\#/\text{in.}$$

The controlling value of a ⅞" rivet 14,430

$$\frac{14{,}430}{3950} = 3.64'' \qquad \text{Use } 3\tfrac{1}{2}'' \text{ pitch.}$$

The exact method as just outlined is cumbersome and lengthy, and an **approximate solution** is more commonly used, which is as satisfactory from a practical point of view. If the total direct stress in the flange were calculated for any vertical section

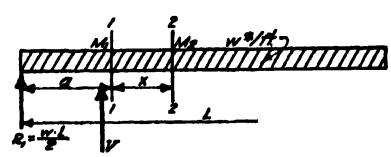


Fig. 106

along the girder, and then at another vertical section, 1" from the first, the difference would be approximately the increase in the flange stress per linear inch. This would also be a lengthy task if carried to a definite result.

For all practical purposes the solution could be made in the following manner. In Fig. 106, let

M_1 = the moment at section 1–1,
M_2 = the moment at section 2–2,
V = the shear at section 1–1.

M_1, M_2 and V bear a definite relation, which is established by the following calculations:

$$M_2 = \frac{w \cdot L}{2}(a + x) - \frac{w(a + x)^2}{2}$$

$$M_1 = \frac{w \cdot L}{2} \cdot a - \frac{w \cdot a^2}{2}$$

$$M_2 = \frac{w \cdot L \cdot a}{2} + \frac{w \cdot L \cdot x}{2} - \frac{w \cdot a^2}{2} - \frac{2\,w \cdot a \cdot x}{2} - \frac{w \cdot x^2}{2}.$$

(The term $-\frac{w \cdot x^2}{2}$ is negligible as it is the square of a very small value.)

$$M_1 = \frac{w \cdot L \cdot a}{2} - \frac{w \cdot a^2}{2}$$

$$M_2 - M_1 = \frac{w \cdot L \cdot x}{2} - w \cdot a \cdot x.$$

But $$V = \frac{w \cdot L}{2} - w \cdot a,$$

or multiplying both sides by x,

$$V \cdot x = \frac{w \cdot L \cdot x}{2} - w \cdot a \cdot x$$

$$\therefore M_2 - M_1 = V \cdot x$$

and $$M_2 = M_1 + V \cdot x$$

The value of p may be substituted for the value of x, if x is assumed to be in inches and the moments are also in inch pounds. Therefore

$$M_2 = M_1 + V \cdot p.$$

But $\frac{M_2}{d_e}$ = the flange stress at section 2–2,

and $\frac{M_1}{d_e}$ = the flange stress at section 1–1.

Substituting, $$\frac{M_2}{d_e} = \frac{M_1}{d_e} + \frac{V \cdot p}{d_e},$$

or $$\frac{M_2 - M_1}{d_e} = \frac{V \cdot p}{d_e}.$$

As p was assumed to be the distance between two rivets, and $\frac{M_2 - M_1}{d_e}$ is the difference in flange stress safely carried by one rivet, then $\frac{M_2 - M_1}{d_e} = R$ (controlling value of one rivet).

$$\therefore R = \frac{V \cdot p}{d_e}$$

and $$p = \frac{R \cdot d_e}{V} \qquad (S\text{–}30)$$

in which p is in inches,
R is in pounds,
d_e is in inches,
and V is in pounds.

This formula may be further explained by a reference to Fig. 107. In this free body diagram of the girder section, two couples balance each other. Therefore, $R \cdot d_e = V \cdot p$, or

$$p = \frac{R \cdot d_e}{V} \text{ (as before).}$$

Illustrative Prob. 60b. Calculate the pitch required for the data of Illustrative Prob. 60a by the approximate method.

$$d_e = 2 \times 30.93 = 61.86''$$

$$p = \frac{R \cdot d_e}{V} = \frac{14{,}430 \times 61.86}{302{,}500} = 2.94''.$$

Use 3″ pitch.

When $\frac{1}{8}\,A_W$ is considered as resisting moment, it may also be used in increasing the rivet pitch. The value of V in the above general formula is reduced by an amount equal to the ratio of the net area of the flange angles and cover

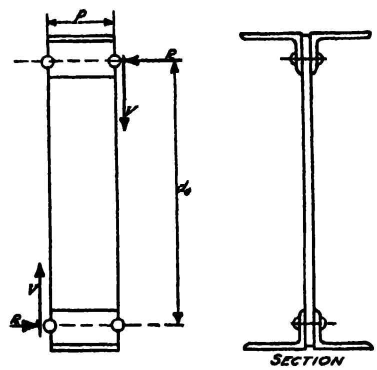


FIG. 107

plates, if used, to the gross area of one-eighth the web plus the net area of the flange angles and plates.* Thus, if this shear is designated by V',

$$V' = V\left(\frac{A_{FN}}{A_{FN} + \frac{1}{8}\,A_W}\right).$$

Then the general formula becomes

$$p = \frac{R \cdot d_e}{V\left(\frac{A_{FN}}{A_{FN} + \frac{1}{8}\,A_W}\right)}. \qquad (S\text{–}31)$$

This will make the value of p larger than before, but only slightly so, and some designers do not use this calculation as the saving in rivets is more than offset by extra time in calculations (Table 29).

Illustrative Prob. 60c. Calculate the pitch required for the data of Illustrative Prob. 60a if $\frac{1}{8}\,A_W$ can be included in the flange area.

$$A_{FN} = 22.96\square''$$

$$A_{FN} + \tfrac{1}{8}\,A_W = 22.96 + 4.64 = 27.6\square''$$

$$p = \frac{R \cdot d_e}{V\left(\frac{A_{FN}}{A_{FN} + \frac{1}{8}\,A_W}\right)} = \frac{14{,}430 \times 61.86}{302{,}500 \times \frac{22.96}{27.6}} = 3.2''.$$

The **approximate method is quite universally used** as the results obtained are on the safe side and close enough. The actual rivet pitch used should be a figure to the nearest $\frac{1}{4}''$ so that the difference is lost in most cases.

There is some variance in the value used for d_e in the formula. The theoretically correct value is of

* The one-eighth web area is not a portion of the flange material which may slide upon the web plate,—hence the reason for this ratio.

course the effective depth of the girder at the point and is advised for use. However, a good argument advanced is that the rivets actually develop the stress and therefore the value to use should be the distance between the gauge lines. If there are two lines of rivets in the flange angle it is customary to use the outside gauge lines, as this distance will be a better approximation to the theoretical effective depth. Another assumption sometimes made is to call the value of d_e the depth of the web plate. The following table for representative girders is interesting as a comparison of the various methods as well as for different values of d_e.*

TABLE 29

VARIATIONS IN RIVET PITCH

	Moment Method Used	Value of d_e Used	Case I	Case II
	Statical Moment (Exact) $q \cdot b = \frac{V \cdot Q}{I}$	(Not used in this method)	4.72″	3.51″
Approximate	Flange to take entire Moment	Theoretical (C. of G. of Flgs.)	3.84″	3.08″
		Dist. betw. rivet lines	3.44″	2.68″
		Depth of web plate	4.05″	3.15″
	$\frac{1}{8}$ Aw used in Flange for Moment	Theoretical (C. of G. of Flgs.)	4.41″	3.32″
		Dist. betw. rivet lines	3.95″	2.89″
		Depth of web plate	4.65″	3.40″

When a **uniform load is applied directly to the top flange** of a plate girder, the rivets must not only resist the longitudinal shear but they also must carry the local shear due to the uniform load. The resultant effect is found by combining the longitudinal shear per linear inch with the uniform load per linear inch (ω). The longitudinal shear per linear inch is $\frac{V}{d_e}$. Therefore, in Fig. 108,

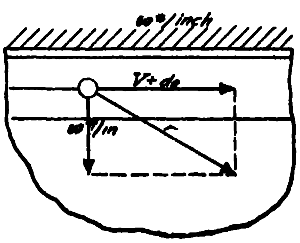


FIG. 108

$$r = \sqrt{\left(\frac{V}{d_e}\right)^2 + (\omega)^2}.$$

As $p = \frac{R}{r}$, $$p = \frac{R}{\sqrt{\left(\frac{V}{d_e}\right)^2 + (\omega)^2}}. \qquad (S\text{-}32)$$

A uniform ceiling load on the bottom chord may be treated in a similar manner. The value of ω may be too small in many cases to have any appreciable effect on the pitch, and once shown to be so, the investigation may be neglected. Concentrated loads on the top flange are carried directly into the stiffeners so that there is no local effect on the rivets.

Illustrative Prob. 60d. Calculate the pitch for the data of Illustrative Prob. 60a if the 8600#/ft. is applied directly to the flange.

$$\frac{8600}{12} = 720\#/\text{in.}$$

Horizontal shear = 3950#/in. (Prob. 60a)

Resultant shear = $\sqrt{(3950)^2 + (720)^2}$ = 4050#/in.

$$p = \frac{14{,}430}{4050} = 3.56'' \quad \text{Use } 3\tfrac{1}{2}'' \text{ pitch}$$

61. Applications of the Formulas.

It has been shown that the pitch of the flange rivets varies directly as the vertical shear. The minimum pitch, therefore, occurs near the supports and the spacing of the rivets gradually increases toward the center of the span. It is impractical to locate each succeeding rivet along the girder at its theoretical spacing, and it is necessary to use average values. This is done by arranging panels and calculating the pitch at the center line of each panel. When stiffeners are used, they form natural panels (Art. 54). If no intermediate stiffeners are required, the tenth points of the span may be used.

The shear, V, is calculated at the point in the span which corresponds to the center-line of the panel and is reduced to V' if one-eighth of the area of the web has been used for moment resistance. The controlling value of the rivet, R, is determined from its strength in shear and bearing, as based upon the specifications (Art. 27). The effective depth, d_e, of the girder cross-section may be calculated for the point under consideration. This value will depend upon the effect of the cover plates which run into the panel. If a cover plate extends over two-thirds or more of the length of the panel, it should be included in the calculations for the effective depth. Otherwise, the calculation should not include that cover plate. The maximum flange stress for the existing section is developed when a cover plate ends, so that the pitch of the rivets required here may be only slightly, if any, larger than the minimum. Where a new cover plate begins, there is a redistribution of stress as the ratio of plate sectional area to the entire flange area changes. It is good practice to use some additional rivets at the minimum pitch for a short distance, say, 1′-0″, either side of the point where the cover plate begins, in both legs of the angle, to insure that the stress may be properly assimilated.

With the above conditions provided for, the pitch, as calculated, will be the average for the panel. The pitch at the end of the panel nearer the

* From an article by R. Fleming in the Engineering Record, June 14, 1923.

reaction should theoretically be less than this value, and at the end toward the center of the span greater than this value; but the number of rivets for the panel, based upon the average pitch, would be the same. In practice the rivets are usually spaced uniformly across the panel, unless it is exceptionally long, in which case the pitch may be arranged in two increments.

Another interpretation of the theory is to connect the flange angles to the web with a sufficient number of rivets to transfer the total shear at any point in a distance equal to the effective depth of the girder at the point, combined with any load that is applied directly to the flange. In Fig. 109 (*a*), V is the vertical shear at the section a–a. As the horizontal shear at any point equals the vertical shear at that point, $H = V$. H must be resisted by the rivets, and if R is the resistance of one rivet, then

$$\frac{V}{R} = n \text{ (the number of rivets required).}$$

But from
$$p = \frac{R \cdot d_e}{V},$$
$$\frac{V}{R} = \frac{d_e}{p} = n.$$

Solving for p,
$$p = \frac{d_e}{n}, \quad \text{or} \quad n \cdot p = d_e.$$

Hence, n rivets at a pitch, p, must be used for a distance equal to d_e.

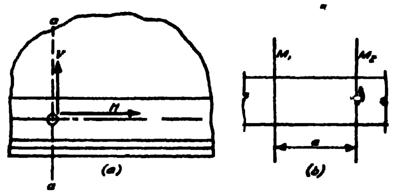


Fig. 109

Still another method is to calculate the moment at one end of the panel, divide by the effective depth at that point and the allowable value of the rivet, and obtain the number of rivets required between that point and the end reaction. Then repeat the operation for the moment at the other end of the panel. The difference between these two cases is the number of rivets required for the panel. The length of the panel divided by this number represents the average pitch. Thus, in Fig. 109 (*b*),

$$\frac{M_1}{d_e \cdot R} = n_1, \quad \text{and} \quad \frac{M_2}{d_e \cdot R} = n_2$$

$$n_1 - n_2 = n \text{ (the number of rivets required in a distance } a\text{).}$$

$$\frac{n}{a} = p.$$

62. Limitations of Rivet Pitch.

SPECIFICATION CLAUSE

Minimum Spacing The minimum distance between the centers of rivet holes shall be three diameters of the rivet; but the distance shall preferably be not less than 3″ for ⅞″ rivets, and 2½″ for ¾″ rivets.

If the pitch calculates less than this value, the rivets must be placed in two lines and staggered. In this case enough stagger must be used to maintain the net section of the flange angles as already planned (Arts. 37 and 48). The adoption of the minimum value is not to be recommended and should be avoided if possible. It will be better to increase the thickness of the web plate, thereby making the value of the rivet larger, or to increase the depth of the web plate, thereby making the effective depth larger and consequently increasing the pitch of rivets, or to use a wider legged angle so that another gauge line may be used. Attention is called at this point to the statement previously given that the thickness of web plate is temporarily determined, contingent upon the possibility of increasing it, to provide a greater spacing of the flange rivets. Many designers calculate the rivet pitch roughly at the outset of the design by using the depth of the web plate as the effective depth and as a guide in selecting the plate girder section. Thus they attempt to avoid such contingencies as above outlined, by considering the influence of the web thickness on the effective value of the rivets.

Where a built-up member is subject to compressive stresses, the parts may tend to wrinkle or bulge if the distance between the rivets is too great.

SPECIFICATION CLAUSE

Maximum Spacing The pitch of rivets shall not be greater than sixteen times the thickness of the thinnest metal connected, nor in any case shall it be greater than 6″.

The rivets in the tension flange are usually limited to the same specification for reasons of symmetry. Since the top flange may have a uniform load applied directly to it, the pitch of rivets theoretically may be less than that in the bottom flange. The spacing of rivets, however, is made the same in both flanges because the theoretical values are only slightly different, and it is a distinct advantage to have the pitch the same on account of the making and using of the same templates. The pitch to be used should then be the value which is the minimum for the condition. The spacing of rivets should be made symmetrical about the center of span, when possible, and as few changes of pitch as are practicable should be made.

The values of the pitch, as theoretically determined, will probably be in decimals. The usual practice is to call for the practical values in multiples of ½″, or at least not closer than multiples of ¼″, under the calculated value. Such rounding off of figures incidentally helps to offset any errors in the calculations.

Prob. 62a. Provide an arrangement of rivets connecting the flange angles to the web plate for the following data:
60 × $\frac{5}{16}$ web plate, 6 × 4 × ½ flange angles, 1-14 × ½ cover plate × 37′-6″, 1-14 × ⅜ cover plate × 25′-6″.

L = 50′-0″, load 5000#/ft.
Stiffeners spaced 6″, 4′-0″, 5′-0″, 5′-0″, etc., from center line of bearing
d_e with no cover plate = 58.5″
d_e with first cover plate = 59.4″. Neglect vertical load per inch. Use approximate method. $\frac{1}{8} A_w$ used as flange material.

Prob. 62b. If the value of d_e is used as the distance between the gauge lines of the flange angles in the above problem, what are the values of the pitches?

63. Graphical Methods.

The pitch of rivets may also be determined by graphical constructions or by the use of diagrams. The moment diagram for a **uniform load** is in the form of a parabola. The minimum variation of moment occurs at the center of the span and the maximum variation at the supports. As explained in Art. 60 the rivet pitch varies in the same manner, for

$$\frac{M_1 - M_2}{x} = V,$$

and

$$p = \frac{V}{d_e}.$$

The procedure of making a graph is as follows:

Calculate the total number of rivets required between the center line of the span and the support, as suggested in Art. 60. Plot one-half the span length to a convenient scale, as AB in Fig. 110, and at the center of span, B, draw an ordinate BC to a convenient scale to represent the total number of rivets calculated. Construct

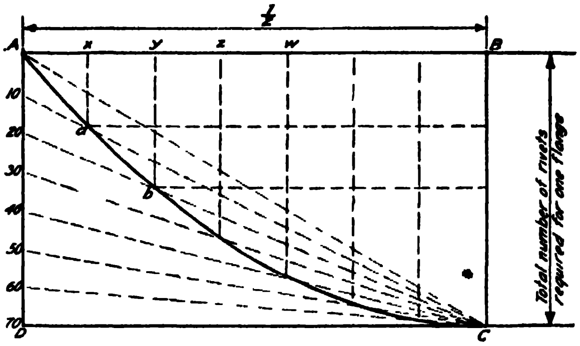


Fig. 110

a parabola on AB with BC as a maximum ordinate, as outlined in Art. 53. Mark the panel points on the diagram, as x, y, and so on. Draw vertical lines from these points to cut the parabolic curve, as xa and yb. Project a and b to the ordinate line BC.* The total number of rivets required between y and A, for example, is 31, and between x and A, is 18. Therefore the number of rivets required in the panel x–y is 31 − 18 = 13. As before, the length of the panel divided by the number of rivets required for that panel will determine the average rivet pitch.

The same method may be applied to conditions where the **loading is not uniform,** by first establishing the moment diagram to a convenient scale and making the total number of rivets required coincide to scale with that of the maximum moment ordinate (for suggestion, see Art. 53), and then proceeding as before. A less exact solution for irregular loading may be made by plotting the maximum moment for the case and constructing a parabola with this value as a maximum ordinate and following the usual procedure through. The average moment diagram will approach such a parabola with small deviations. In special cases, such as for concentrated loads at the third-points of the span, the parabola so constructed would be considerably different from the real moment diagram. It is therefore recommended that the latter diagram should be made for all cases, as the time spent in showing the true variation of moment will produce more accurate values of the rivet pitch.

Many **diagrams** are used to obtain the rivet pitch without plotting each time. Some of these are time savers and of sufficient accuracy to warrant their use. One should make sure that the diagram always corresponds with the conditions of the problem. Figure 111 shows such a typical diagram.* The following describes its use:

"In the accompanying diagram for the rapid determination of rivet pitch in plate girders, two lines of ⅞″ rivets are used as basis of calculation, so that the diagram applies to 6″ × 6″ flange angles. It is based on a bearing value of 24,000 lbs. per sq. in.

The diagram solves the equation $p = \frac{d \cdot V}{S}$ in which p is the rivet pitch in inches, d the depth between centers of gravity of flange rivet lines in inches, V the value of one rivet and S the shear, the latter two being in the same unit (pounds or thousands of pounds). The value of V obviously varies with the web thickness, but it is convenient to remember that the double shear value of a ⅞″ rivet is substantially the same as its value in bearing on an 11/16″ web.

"To illustrate the use of the diagram, suppose a girder with 48 × ⅜″ web carries a shear of 100,000 lbs. In the lower scales, for web thickness, select the line marked ⅜, and go vertically from the value 100 (representing 100,000 lbs.) on this line to the horizontal line representing 48″ depth of the web. The intersection lies between the sloping lines 3 and 3¼, and the required pitch therefore is 3″.

"When only one line is used, the depth for entering the diagram may be taken 2″ greater than the actual depth and the results will be very close to correct. If the girder, for example, had 6″ × 4″ flange angles, 50″ may be used for the depth, and the pitch will be found to be 3¼ inches.

"Obviously, the diagram is not applicable to railway deck girders or others that carry concentrated loads directly on the flanges." *

* Article by R. Smilie, Engineering News Record, July 22, 1920.

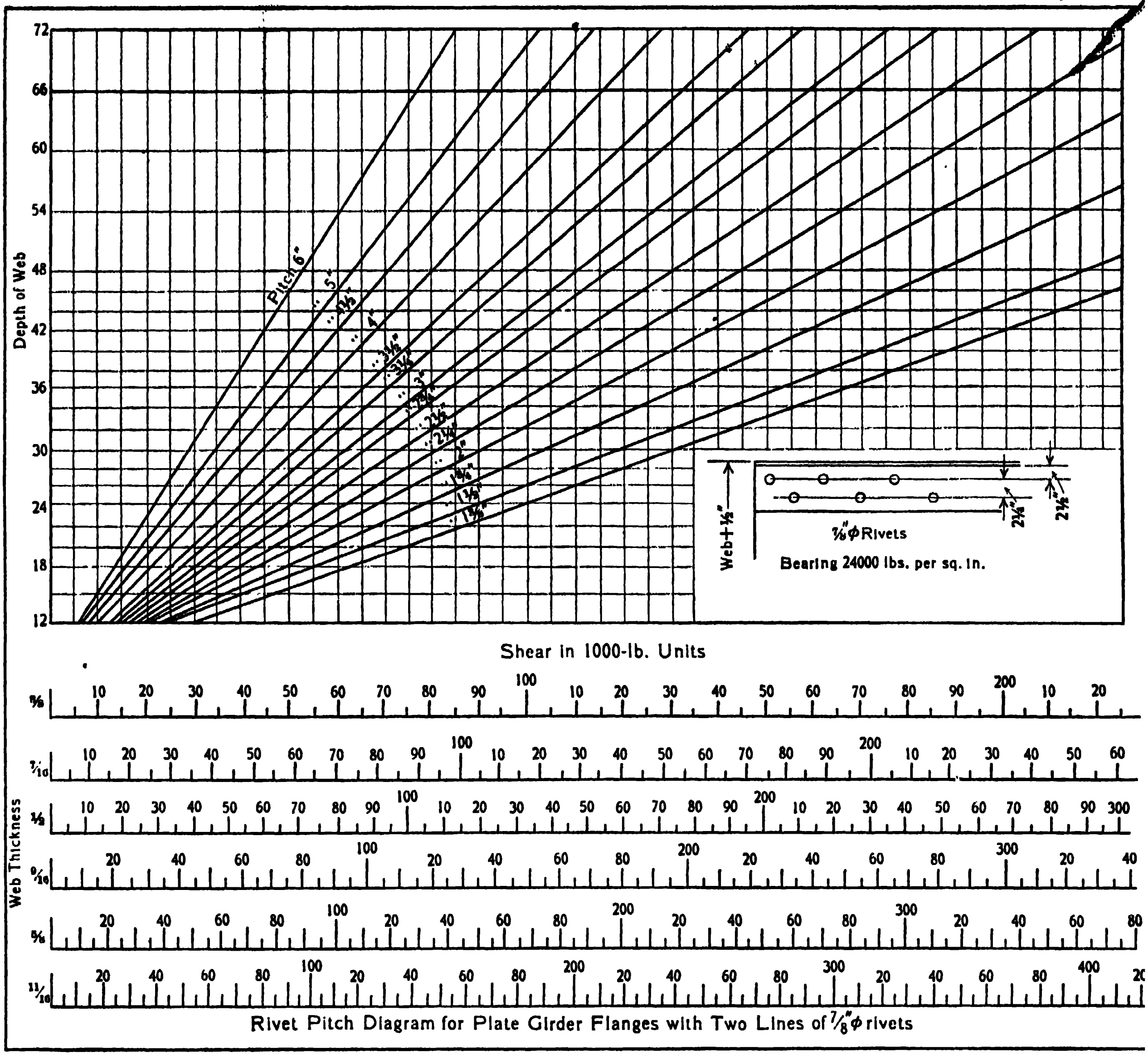


Fig. 111

Section 5g

RIVETS IN THE COVER PLATES AND STIFFENERS

64. Methods Used for the Determination of the Pitch of Cover Plate Rivets.

The rivets in the horizontal legs of the flange angles passing through the cover plates transmit only a part of the flange stress. Consequently they may have a larger pitch than that of the rivets in the vertical legs of the angles at the same point. The general action of the longitudinal shear has already been discussed (Art. 60). This force would create a tendency of the cover platès to slide on the flange angles, which tendency must be resisted by the rivets which are to hold these parts together. The controlling value of the strength of the rivets will be that of single shear or bearing, depending upon the thickness of the cover plates or angles.

The **exact method** will involve considering the statical moment of the cover plate (or plates) only, about the neutral axis, when using the formula given before as:

$$q = \frac{V \cdot Q}{b \cdot I}.$$

Here the value of b is the width of the plane on which the cover plates rest. This is always the sum of the widths of the outstanding legs of the flange angles. The intensity of horizontal shear at such a plane is q and the shear per linear inch is

$$q \cdot b = \frac{V \cdot Q}{I}.$$

If the shear per linear inch is x pounds, and the controlling value of the rivet R pounds, the pitch is

$$P = \frac{x}{R} \text{ (as before, Art. 60).}$$

Since the intensity of longitudinal shear is 0 at the extreme fibre, that in the cover plate is rapidly approaching 0, so that the value of $q \cdot b$ is considerably less than that when the flange angles are considered.

Illustrative Prob. 64a. Determine the pitch of the cover plate rivets for the following data:

66 × $\frac{9}{16}$ web plate, flange angles 8 × 8 × $\frac{7}{8}$, first cover plate 18 × $\frac{3}{4}$. Shear at end of cover plate toward reaction = 268,000#. Distance b. to b. L^s = 66.5. $\frac{7}{8}''$ rivets. Statical moment of plate about neutral axis

$$18 \times \tfrac{3}{4} \times \left(\frac{66.5}{2} + \frac{0.75}{2}\right) = 463''^3.$$

Moment of inertia of whole section at point about N. A.

$$66 \times \tfrac{9}{16} \text{ Pl.} = \frac{b \cdot d^3}{12} = \frac{0.5625 \times (66)^3}{12} = 13{,}470$$

I of 4 L^s = 4 × 79.6 = 320

$A \cdot d^2$ for 4 L^s = 13.23 × 4 × $\overline{30.93}^2$ = 50,600

(I of cover plate about own axis neglected)

$A \cdot d^2$ for C.Pl. = 18 × $\frac{3}{4}$ × $\overline{33.62}^2$ × 2 = 31,200

Total = 95,590$''^4$

$$b \cdot q = \frac{V \cdot Q}{I} = \frac{268{,}000 \times 463}{95{,}590} = 1300\#/\text{in.}$$

Single shear $\frac{7}{8}''$ rivet = 7220#. Two rivets in a transverse line. Hence

$$p = \frac{2 \times 7220}{1300} = 11.1''.$$

The exact method, however, is too irksome to use in the average case, and an **approximate solution** is generally considered as **satisfactory.** A formula may be derived by taking the one used for the pitch of rivets in the vertical legs of the flange angles and modifying it by the ratio of the total net area of one flange, A_{FN}, to the total net area of the cover plates, A_{CN}. Such a step may be made on the assumptions that the horizontal shear is uniformly distributed over the entire flange section, and that the cover plate is taking its share of such a force according to the ratio of its area to the entire flange area. Thus

$$p = \frac{R \cdot d_e}{V} \text{ (Art. 60).}$$

For cover plates,

$$p = \frac{R \cdot d_e}{V} \times \frac{A_{FN}}{A_{CN}}.$$

However, there probably will be more than one rivet in a transverse line across the cover plate and there are usually two. Hence, if n = the number of such rivets, the formula becomes

$$p = \frac{n \cdot R \cdot d_e}{V} \times \frac{A_{FN}}{A_{CN}}. \qquad (S\text{–}33)$$

Illustrative Prob. 64b. Determine the pitch of rivets in the cover plate in Illustrative Prob. 64a by the approximate method. d_e = 65.5″.

The net area of the flange with one cover plate = 39.6□″

The net area of the cover plate = 12.0□″

$$p = \frac{n \cdot R \cdot d_e}{V} \times \frac{A_{FN}}{A_{CN}} = \frac{2 \times 7220 \times 65.5}{268{,}000} \times \frac{39.6}{12.0}$$

p = 11.6″.

Compare with Illustrative Prob. 64a.

65. Practical Requirements for Cover Plate Rivets.

When there is more than one cover plate, naturally the grip of the rivets is increased, and consequently the rivets are not quite as efficient. The outer plates tend to slide on those beneath them, so that a rivet has to resist this action as well as that of the inner plate tending to slide on the flange angles. Theoretically these two amounts of horizontal shear should be calculated separately and then added and the rivet pitch based on this sum. Such a degree of accurate calculation is unwarranted, and the following specification is advised.

SPECIFICATION CLAUSE

Reduction of Pitch When more than one cover plate exists, the pitch as theoretically calculated shall be reduced 15% for each cover plate more than one.

Maximum requirements for the pitch of rivets in cover plates are generally covered by specification clauses. Where two or more plates are in contact, rivets should not be spaced greater than 12″ apart in any direction. This requirement assists in holding the plates well together and prevents them from wrinkling or buckling when subjected to compressive forces.

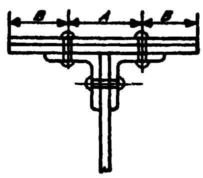
SPECIFICATION CLAUSES

Maximum Pitch The maximum pitch of rivets in a direction parallel to the line of stress shall not be greater than 16 times the thickness of the thinnest plate, nor in any case greater than 6″.

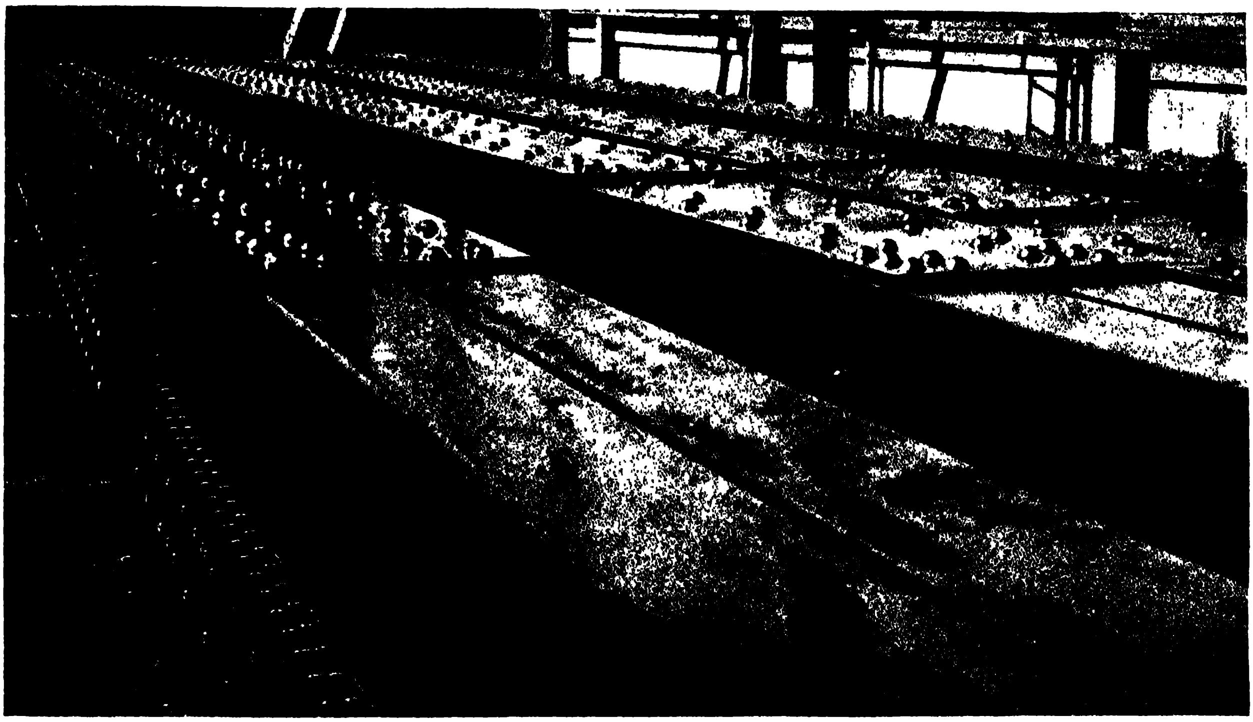
Distance between Opposite Lines The distance A (Fig. 112), perpendicular to the line of stress, shall not be greater than 32 times the thickness of the thinnest plate.

Edge Distance The distance B (Fig. 112) shall not be greater than 8 times the thickness of the thinnest plate, plus 1″, nor, in any case, greater than 5″.

Some structural companies allow the maximum pitch to be increased from 6″ to 8″ for $\frac{7}{8}$″ and 1″ rivets. This corresponds to a reaction in recent years that cover plate riveting has been excessive. When the flange angles each have two available



(a)



(b)*

FIG. 112

gauge lines in the outstanding legs it is possible to use four lines of cover plate rivets. This is usually not necessary, especially if the girder is to be encased in concrete, and two lines of staggered rivets, one either side of the web plate, are ordinarily sufficient. Figure 112 (b) shows a typical view of cover plate rivets.

Pitches as determined by the usual formula will result in values with decimals. These should be reduced to the nearest $\frac{1}{4}$″ and preferably to the nearest $\frac{1}{8}$″ below the theoretical values, as explained before (Art. 62). A common procedure is to locate the end rivets $1\frac{3}{4}$″ from the ends of the cover plates to provide ample edge distance in case the plates should underrun in mill shipping lengths.

In order to drive the rivets and not have them foul each other, and to maintain the net section, rivets in the cover plates, in the majority of instances, must stagger with those in the vertical legs of the flange angles. Rivets on the outside gauge lines may be driven in the same vertical line with those in the vertical legs of the angles in many cases. Therefore, it is expedient to keep cover plate rivets on the outer gauge lines when possible. In detailing, the plan of the girder should be laid out to see if any rivets in the cover plates foul the rivets in the flanges or the stiffener angles themselves.

The first cover plate must have its full strength developed by rivets before the second cover plate is reached, and so on, and the top plate must have its full strength developed before it reaches the point of maximum moment. Some designers go to the extent of developing the full strength of the cover plates by using the minimum allowable pitch at the ends, and then using the maximum allowable pitch elsewhere. This method requires more rivets than necessary in the majority of cases, and it may

* Courtesy of New England Structural Co.

reduce the net section used in the design by necessitating an insufficient stagger of rivets.

Prob. 65a. Determine the pitch of the rivets in the first cover plate for the following data:

Use approximate solution. $\frac{7}{8}''$ rivets
$60 \times \frac{5}{16}$ web plate ($\frac{1}{8}\ A_W$ used in flange)
$6 \times 4 \times \frac{1}{2}$ flange angles, net area $= 7.50\square''$
$14 \times \frac{1}{2}$ cover plate, net area $= 6.00\square''$
Shear at end of plate $= 94{,}000\#$.

How many rivets are necessary to develop the tensile strength of the plate?

66. Rivets in Stiffeners at Concentrated Loads.

The number of rivets required to make a stiffener function properly is a simple calculation, being that of dividing the concentrated load by the controlling value of the rivet, or

$$n = \frac{P}{R}.$$

Such a calculation is based upon the assumption that each rivet takes its share of the load. In general, the rivets passing through the flange angles should not be counted upon to develop the load, as they may be fully stressed by flange action. When the stiffeners under consideration are at or near the supports, these rivets may be counted upon if necessary, as the flange stress at such points is relatively small. When the number of rivets is known, the arrangement may be obtained by spacing them equally in the available height. Such spacing should not exceed the usual specified maximum (Art. 25).

Fillers have already been mentioned as necessary to make up for the difference in thickness of the web plate and flange angles. These are classified in two groups as **loose fillers** and **tight fillers.** A loose filler is a bar which has no other connection to the web plate except the rivets which pass through the stiffener, while a tight filler is riveted to the web plate by one or more independent rows of rivets. When rivets pass through a loose filler the number should be increased as there is some bending on the rivets between the point where they pass through it and that portion from which they receive their load. A common rule of thumb is to add 10% to the number otherwise required, for each loose filler. A tight filler can be used to advantage where the number of rivets required for the load would make the spacing in the stiffener too close. A value equal to that which the rivets fixing the filler can carry, may be taken from the web plate into the filler and thence carried by the filler into the stiffener by its connecting rivets. The rivets fixing the filler will have bending on them in addition to the shear. Rather than figure the combined stress on these rivets, for a detail of this kind, it is usually sufficient to make a liberal allowance. Some designers, therefore, double the number of rivets required, to allow for fixing the filler.

If a concentrated load occurs very near a support, reinforcing web plates may be used and extended beyond the concentration a distance sufficient to develop the plate by rivets. Such a plate is in reality a tight filler and the design is accomplished as before.

Illustrative Prob. 66a. If a pair of $6 \times 3\frac{1}{2} \times \frac{3}{8}$ ∟s acting as stiffener at a concentrated load of 65,800# are riveted to a $66 \times \frac{7}{16}$ web plate, how many $\frac{7}{8}''$ rivets are required?

Double shear $= 14{,}440\#$
Enclosed bearing on $\frac{7}{16}''$ web $= \frac{7}{8} \times \frac{7}{16} \times 30{,}000 = 11{,}500\#$

$\frac{65{,}800}{11{,}500} = 6.$ 2 loose fillers $6 \times 1.2 = 7.2$ required
(10% for each loose filler)
$66 - 5''$ (for gauges) $= 61''$ out to out rivets.

$\frac{61}{6} = 10$ spaces.

Use 11 rivets 6″ o.c. on account of specification.

Prob. 66b. How many $\frac{3}{4}''$ rivets are required for a pair of $5 \times 3\frac{1}{2} \times \frac{3}{8}$ angles carrying a load of 80,000#? Web plate $60 \times \frac{3}{8}$. What spacing should be used?

67. Rivets in Stiffeners at End Bearings.

The detail design required for obtaining the number of rivets necessary in stiffeners at the supports of a girder is the same as that for any other concentrated load, although special features may develop. Some designers prefer an alternate

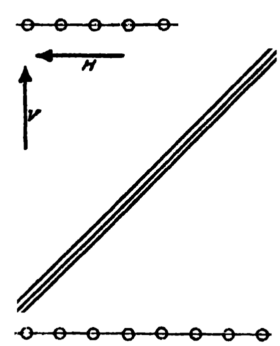


FIG. 113

method of design, namely, by making the number of rivets in the end stiffeners equal to the number required to connect the flange angles to the web plate in a distance equal to the effective depth of the girder. The reasoning used is that the diagonal stress caused in the end panel is the result of a horizontal shearing force, as H in Fig. 113, and a vertical shearing force, V. If x rivets are required in the distance AB, then the same number are required in a distance BC. When there are several

pairs of stiffeners required for the end reaction, a one-piece tight filler under them is cheaper than loose fillers under each. If a girder is to frame into a column the better detail from a standpoint of reliability and erection is to use a seat angle with stiffeners under it. When one girder frames into another and there is not room for a seat angle, the tight filler may be used to good advantage.

Prob. 67a. How many $\frac{3}{4}''$ rivets are required in the stiffeners in Fig. 113 if the end reaction, R, is 200,000#? Web plate $60 \times \frac{1}{2}$. Stiffener angles $5 \times 3\frac{1}{2} \times \frac{1}{2}$. Make a sketch showing the spacing to be used.

68. Rivets in Intermediate Stiffeners.

The number of rivets to place in intermediate stiffeners cannot be determined by any definite calculation. The arrangement is governed principally by the number and spacing of rivets in other stiffeners.

SPECIFICATION CLAUSE

Maximum Spacing The maximum spacing of rivets in intermediate stiffeners shall not exceed 16 times the thickness of the angle, nor be greater than 6″.

The spacing of rivets from top to bottom in the stiffeners should be uniform across the girder for the sake of symmetry and also to make multiple punching possible. Some rivets may be omitted in such stiffeners as compared with those in other stiffeners, providing the maximum spacing just specified is not exceeded. The reason why the rivets in intermediate stiffeners cannot be more positively located is because the load coming upon them is not definite. However, the common practice has proven to be satisfactory.

Section 5h

SPLICES

69. General Requirements.

There are limiting lengths of plates and angles as governed by rolling mill practice (Art. 7) so that it may become necessary to splice the web plates, sometimes the cover plates, and in special cases, the flange angles of long plate girders. If the part under consideration can be obtained in one piece, it should be used to avoid splicing. However, local fabricators may use the available lengths of their stock material. In such a case the location of the splice is a detail handled by the fabricator and not by the engineer. If the length of the girder is such that the maximum available lengths of the parts will not allow the girder to be fabricated without splices, the engineer may show where he prefers the splices to be located and in certain instances he may design the required splice. In either of the above cases, the design involves the same procedure.

Many specifications require that the splice shall be as strong as the original parts spliced. A splice, particularly in as large a member as a plate girder, is a critical point, regardless of where it is located, because there are many factors which may weaken its strength, such as loose rivets, misalignment, and the like. To supply area enough to compensate for the area cut may be an excessive provision in many cases, for the maximum moment and the maximum shear cannot occur at the same point, and if the splice is located at a proper point, neither the moment nor shear at such a point will be a maximum. Nevertheless, girders may be loaded differently than the original loading diagram used in the design because of a different occupancy of the building, concentrations, and so on, and it would not be desirable to have a girder overstressed on account of its splices. In special instances when the specifications permit, a splice may be designed for the actual moment and the actual shear which occur at the splice line. Such practice should be limited to conditions which are positive.

The following suggestions are of value in designing splices:

(1) Locate a splice where there is an excess of material and not at a point of maximum stress.

(2) Do not use splices if the material may be obtained in one length.

(3) No two parts should be spliced within 2′-0″ of each other, and preferably the distance should be greater.

(4) The rivets should be arranged as closely as convenient to make the transfer of stress occur in a minimum distance. The spacing, however, must not violate the requirements of the net section and must provide the necessary clearance for driving rivets.

(5) No allowance should be made for the abutting edges of the parts spliced.

(6) A desirable point to locate a splice is just inside the end of a cover plate, for at such a point, practically all the area of that cover plate is in excess. This excess metal will aid in supplying any deficiency in strength the splices might have.

70. Types of Tension and Shear Splices.

When plates are spliced directly or when they are used to splice rolled shapes, the joints which may be used are classed as:

(1) lap joints.

(2) butt joints $\begin{cases} (a) \text{ plate on one side only, and} \\ (b) \text{ plates on both sides.} \end{cases}$

Of these types (2 *b*) is by far the most commonly used, and in fact, it is always used when possible, the others being exceptions.

The **lap joint** is seldom allowed in structural work, first, because it presents a poor appearance and makes bad details, and secondly, and principally, because the forces are eccentric, by an amount equal to the center to center distance between the plates in cross-section, or l in Fig. 114 (*a*). The direct stress equals the load on the plate divided by its area, or

$$\frac{P}{b \cdot t} = s_1.$$

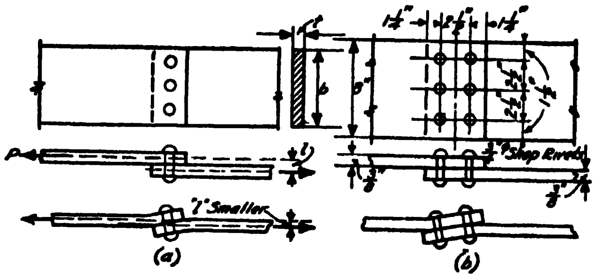


FIG. 114

The external moment, $M_e = P \cdot e, = P \cdot t$ if the plates are equal in thickness. The resisting moment is

$$M_r = s_2 \cdot \frac{b \cdot t^2}{6},$$

in which s_2 = the maximum allowable fibre stress.

Equating $M_e = M_r$,

$$P \cdot t = s_2 \cdot \frac{b \cdot t^2}{6} \quad \text{or} \quad s_2 = \frac{6P}{b \cdot t} = 6\,s_1.$$

If there were no deformation, the unit bending stress would be six times the unit direct stress, which obviously controls the design. As the joint deforms, the eccentricity becomes smaller, as shown in the figure, and hence the moment smaller, but the rivets would certainly be in tension. Figure 35 (*d*) shows a typical failure of this kind.

Illustrative Prob. 70a. Considering the joint in Fig. 114 (*b*), what force can it safely withstand in tension?

$$\text{Single Shear per rivet} = 12{,}000 \times 0.4418 = 5300\#$$
$$\text{Unenclosed Bearing per rivet} = 24{,}000 \times \tfrac{3}{4} \times \tfrac{3}{8} = 6750\#$$
$$5300\# \text{ controls}$$
$$6 \times 5300 = 31{,}800\#, \text{ allowable shear}$$

$$\text{Allowable direct tension} = [8 \times \tfrac{3}{8} - (3 \times \tfrac{7}{8} \times \tfrac{3}{8})]\,16{,}000 = 47{,}800\#$$
$$M = P \times \tfrac{3}{8}.$$
$$\tfrac{1}{6} \times 8 \times (\tfrac{3}{8})^2 \times 16{,}000 = P \times \tfrac{3}{8}.$$
$$P = 8000\#, \text{ allowable force.}$$

From this it should be obvious that such a joint is not efficient.

The **butt joint with one splice plate** is in reality two lap joints in tandem, and the same objections hold true (Fig. 115). The path of the stress is from the main plate on one side to the rivets, to the splice plate, to the rivets on the other side, and back to the main plate on the other side. This action induces a large bending moment, and the design is similar to that of lap joints.

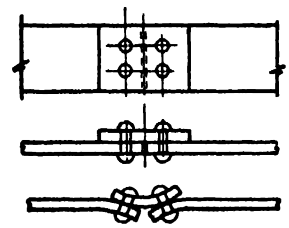


FIG. 115

The **butt joint with a splice plate on each side,** as in Fig. 116 (*a*), is the one most commonly used for structural members. Due to the fact that there is a plate each side of the main plate, no bending is theoretically developed. The rivets may fail in double shear or in bearing on the main plate, or on the combined thicknesses of the splice plates. For

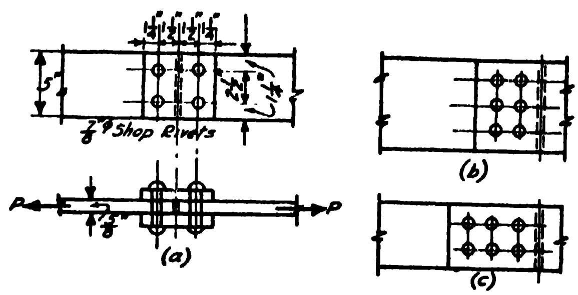


FIG. 116

this reason each of the splice plates should have a thickness equal to, or greater than, one-half the thickness of the main plates. In almost all compression calculations, no allowance is made for the main plates butting against each other, and in fact, each of the plates is generally cut $\frac{1}{4}''$ short of the center line of the splice, the rivets carrying the stress across the joint.

Illustrative Prob. 70b. What force can the joint shown in Fig. 116 (*a*) safely withstand in tension?

$$\text{Double shear per rivet} = 2 \times 0.6013 \times 12{,}000 = 14{,}430\#$$
$$\text{Bearing } \tfrac{5}{8}'' \text{ plate per rivet} = 24{,}000 \times \tfrac{5}{8} \times \tfrac{7}{8} = 13{,}130\#$$
$$13{,}130\# \text{ controls}$$
$$13{,}130 \times 2 = 26{,}260\# \qquad \textit{Ans.}$$
$$\text{Allowable tension on net section} = 16{,}000\,[5 \times \tfrac{5}{8} - 2\,(1 \times \tfrac{5}{8})] = 29{,}900\#$$

The splice or butt plates should be $\frac{5}{16}''$ or $\frac{3}{8}''$ thick.

For wide plates, it may not be convenient to consider the entire width at once. The plate may

be divided into a number of equal strips each of which includes a group of rivets, and these being designed properly, the continuous joint will be satisfactory. In some cases, judgment will determine the arrangement of the rivets. In Fig. 116 (*b*), the net section is smaller than in (*c*), while the strength of the rivets is the same in bearing and shear. Type (*c*) requires a slightly larger splice plate but it provides better side distance for the rivets.

The **arrangement** shown in Fig. 117 is more or less **theoretical** and it is not commonly resorted to in structural joints. The net section at W–W has one hole out. Before the main plate can tear across X–X, the rivet at W–W must fail by shear or bearing. This offsets the weakening of the main plate by one rivet hole so that section X–X is really only weakened by one hole, and, therefore, it could be considered as strong as the section W–W. The same reasoning can be applied to Y–Y for no failure can occur here until the rivets at W–W and X–X fail. The splice plate section, however, is reduced by three holes and consequently must have a thickness greater than one-half the thickness of the main plate. If the plate in Fig. 117 were $8'' \times \frac{1}{2}''$ and the rivets were $\frac{7}{8}''$, the thickness of one splice plate would be

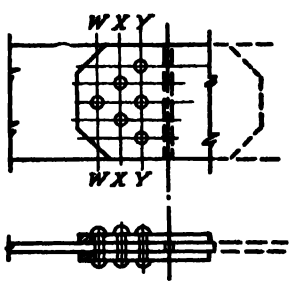


Fig. 117

$$t = \frac{16{,}000\,(8 \times \frac{1}{2} - 1 \times \frac{1}{2})}{2\,(8 - 3 \times 1) \times 16{,}000} = 0.35'',$$

or $\frac{3}{8}''$ splice plates would be required.

In certain joints, it becomes necessary to **determine** what the **maximum condition of stress** is. Thus in Fig. 118, the shears are as follows:

Fig. 118

Between plates 1 and 2, $V = 60{,}000$
2 and 3, $V = 10{,}000$
3 and 4, $V = 10{,}000$
4 and 5, $V = 40{,}000$

The maximum condition of rivet stress must also be investigated. From Table 24, for $\frac{3}{4}''$ field rivets,

Bearing on $\frac{5}{16}''$ plate = 4690#
$\frac{3}{8}''$ plate = 5630
$\frac{7}{16}''$ plate = 6560
$\frac{1}{2}''$ plate = 7500
Single shear = 4420

The number of rivets required in each instance then is,

$\frac{1}{2}''$ plate, $\frac{110{,}000}{7500} = 14.7.$ $\frac{7}{16}''$ plate, $\frac{100{,}000}{6560} = 15.3$

$\frac{3}{8}''$ plate, $\frac{70{,}000}{5630} = 12.4.$ $\frac{5}{16}''$ plate, $\frac{50{,}000}{4690} = 10.7$

Shear, $\frac{60{,}000 \text{ maximum}}{4420} = 13.6.$ Use 16-$\frac{3}{4}''$ field rivets.

Prob. 70c. Determine the safe strength of the joint shown in Fig. 119 (*a*).

Prob. 70d. What is the maximum safe tension that the joint in Fig. 119 (*b*) can withstand?

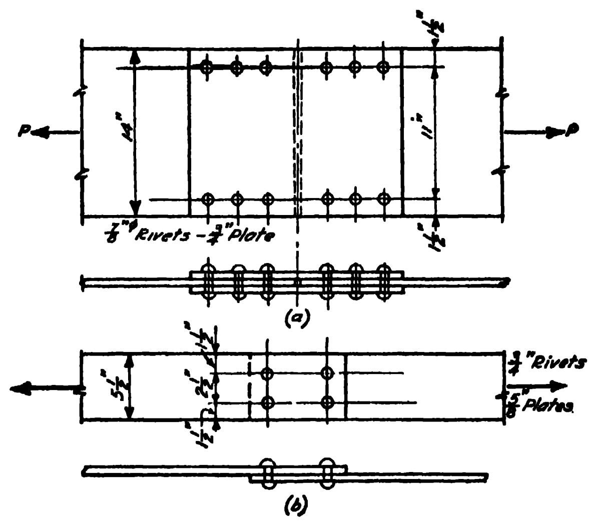


Fig. 119

71. Types and Requirements of Web Splices.

A web splice should be strong enough to equal the capacity of the web plate in shear, and also to withstand any proportional part of the moment that the web has been designed to resist.* The latter value is usually expressed by

$$M_W = M\left(\frac{\frac{1}{8}A_W}{A_{FN} + \frac{1}{8}A_W}\right). \qquad (S\text{–}34)$$

If the design specifications did not allow any portion of the web plate to be counted upon as flange material, the usual procedure is to design the splice for the shear capacity of the web only. This point must be positively known before a web splice is designed.

It is desirable to have the web plate in the minimum number of pieces, but for very long girders it may be more economical to use splices. The width

* Some designers neglect the portion of the moment assigned to the web because they believe that web splices are none too efficient at best and that web splices are unsatisfactory in resisting moment. However, if designed for the moment the web resists, the splice is safer than if not so designed. Some specifications require that a web splice shall be designed for moment and shear even though the web has not been counted upon for moment, as the additional strength of neglecting the bending value of the web is lost if the web is not spliced for moment. Even if the web plate is not designed to take moment, it will, nevertheless, actually carry a portion of the bending.

of the splice plate must be sufficient to accommodate the rivets and maintain proper edge distances for them. It is better not to use the minimum spacing of rivets if it can be avoided, but to use 3″ or 4″ on centers instead. This is done to maintain the equivalent of the net section of the web plate (Art. 45).

The usual types of splice plates are shown in Fig. 120. A plate should be used on both sides to keep the joint concentric and to increase the efficiency of the rivets. There should be at least two lines of rivets either side of the splice line. This will prevent any tendency of the joint to rotate, which might occur with one line of rivets. If a rivet in one line started to fail, the rivet in the other line would help to relieve it. The standard clearance dimension is $\frac{1}{4}''$ from the edge of the web plate to the splice line each side, to allow for any irregularity of the edges of the web plate as sheared, as shown at A in Fig. 120 (a). When the splice is not under a beam connection, $\frac{1}{4}''$ is also allowed top and bottom for clearance, as shown at B, for the width of the flange angles may overrun. Type (*a*) is the most common and it should be used if possible. It is used for small girders and also where loading conditions are not too heavy, and particularly when no moment is considered to be carried by the splice. In cases where the moment to be carried by a splice is relatively small, this type may still be used, if the rivets can be conveniently arranged. Since this type is the simplest, others are used only when too many vertical lines of rivets would be required. Not over three or four lines either side of the splice should preferably be used in this arrangement.

In certain cases it may be more convenient to locate a splice directly under a stiffener. The stiffener will help the splice if its center line is placed under the stiffener as shown in Fig. 120 (*b*). Two alternate details may be used. In one, the splice plate can serve the additional purpose of being the filler for the stiffener. Here the splice plate will have to be the thickness of the flange angles. This may be in excess of the material required for the splice. The other alternate is to use a separate filler under the stiffener, the thickness of which is the difference between the thickness of the flange angle and that of the splice plate. In this case extra rivets should be added as specified in Art. 66. In either type (*a*) or (*b*) the rivets in the vertical legs of the flange angles for a short distance, say, 1′-0″, on each side of the splice line may be considered as a part of the group of splice rivets, as they really help to splice the web because of their attachment to the flange angles. If such rivets are counted upon, the longitudinal shearing stress combined with that for the splice should not exceed the allowable for the rivets.

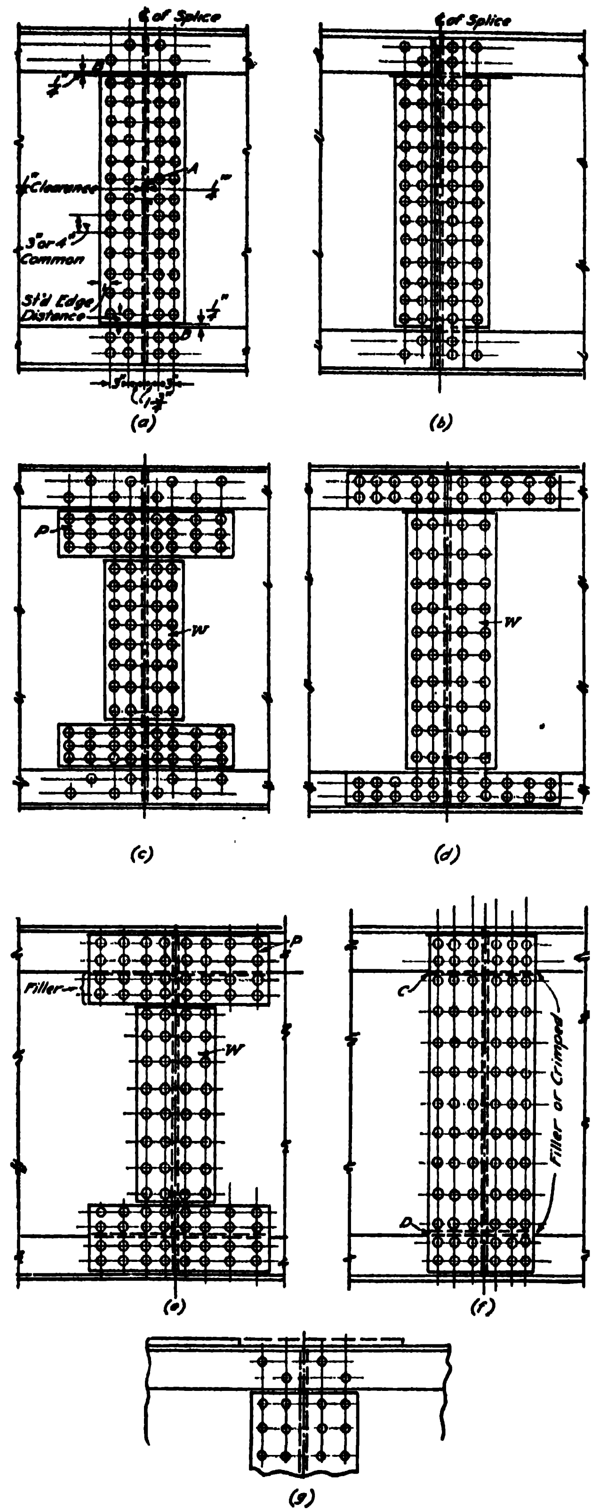


FIG. 120

When types (*a*) and (*b*) are not satisfactory or there is a considerable amount of moment involved, types (*c*), (*d*), (*e*) or (*f*) may be used. The rivets are more efficient in resisting moment because a

larger number of them will act with greater lever arms than in (*a*) and (*b*). The total number of rivets will be less and therefore less plate surface is required. The farther the rivets are removed laterally from the splice line, the less efficient they are in making a transfer of stress. There should be a practical limit for the number of lines in the plates, *P*, for this reason. This is usually made 5 or 6 lines. Type (*c*) is preferable to (*d*) in that it does not encroach upon the flange angles, but (*d*) obtains a better splice on the web plate when its whole depth is considered. Type (*d*) may be used when a sufficient number of rivets cannot be arranged in (*c*). In general, three rows of rivets in the plates *P* in the figure (*c*) should be the limit, because more than that will make the splice approach types (*a*) and (*b*). For the total number of rivets used, the majority in the latter case would not be acting at lever arms to make them efficient in resisting moment. When web splice plates are used on the vertical legs of the flange angles, as in (*d*), they should be designed to splice the portion of web covered by the flange angles. Some designers use a method of designing the plates *P* for the moment and the plates *W* for the shear. This method does not seem as logical as to consider the group as a unit and design it for both stresses combined. In types (*e*) and (*f*), the plates should be strong enough to safely provide for the horizontal shear at the edge of the flange angles. Type (*f*) shows an alternate detail, in which the splice plates may be crimped as shown at *C* or fillers used, as shown at *D*. Crimping plates is expensive but the use of additional fillers increases the grip of the rivets and consequently lessens their efficiency. However, crimping plates is not recommended. If the splice is to be located near the end of a cover plate, additional protection is gained by running the cover plate by the splice, as suggested by the dotted lines, in Fig. 120 (*g*).

72. Web Splices to Resist Shear Only (Art. 70).

When the web plate is designed to resist no moment, the calculation of the splice is comparatively simple. The two splice plates must have a net area at least equal to the net area of the web plate. The splice plates must also be riveted to the web plate on either side of the splice line with a sufficient number of rivets to develop the shear value of the web plate. The number of rivets required will be that found by dividing the shear value of the web by the controlling value of the rivet as determined by bearing or double shear. The latter suggestion is based upon the assumption that each rivet takes its share of the load. As a matter of fact, the rivets near the neutral axis are carrying more than those near the upper and lower edges of the splice, for the actual intensity of shear is larger there. However, the assumption is reasonable because only a few rivets are stressed more than the average value, while the majority are stressed at the average or below, so that the group as a whole is satisfactory.

Illustrative Prob. 72a. Design a splice for a $60 \times \frac{3}{8}$ web plate to resist shear only. 6×6 flange angles. $\frac{3}{4}''$ rivets.

b. to b. $\llcorner$s $= 60 + \frac{1}{2} = 60.5''$
$60.5 - 2 \times 6 - 2 \times \frac{1}{4} = 48''$ height of splice plate
$A_{WN} = 60 \times \frac{3}{8} \times \frac{3}{4} = 16.9\square''$
$16.9 = 48 \times t \times 2 \times \frac{3}{4}\ t = 0.23''$

Use 2-$\frac{5}{16}''$ plates for practical reasons.

Double shear $= 10{,}600\#$
Bearing on web $= 24{,}000 \times \frac{3}{8} \times \frac{3}{4} = 6750\#$
$16.9 \times 12{,}000 = 203{,}000\#$ shear capacity of web
$\frac{203{,}000}{6750} = 30$ rivets.

Use two vertical lines each side of splice.

$15 \times 3'' = 45''$. $45 + 2 \times 1\frac{1}{2}$ (edge distance) $= 48''$
Use 32 rivets, 16 in a line, 3″ o.c. $1\frac{1}{2}''$ edge distance
Space laterally $1\frac{1}{2}''$ edge distance, 3″, $3\frac{1}{2}''$, $1\frac{1}{2}''$ edge distance. (See Fig. 120 (*a*)).

Use 2-$12 \times \frac{5}{16}$ Pls. $\times$ 4′-0″.

Prob. 72b. Design a splice to resist shear only for a $48 \times \frac{3}{8}$ web plate. 6×4 flange angles. $\frac{7}{8}''$ rivets.

73. Web Splices to Resist Both Shear and Moment.

If a portion of the gross area of the web plate has been considered as flange area, or the specifications require it, the web splice must be designed for moment as well as shear. In such a case, the splice plates and rivets must be sufficient to carry not only the shear value of the web, but its proportional part of the total bending moment. The latter value is expressed by

$$M_W = M\left(\frac{\frac{1}{8}A_W}{A_{FN} + \frac{1}{8}A_W}\right) \qquad \text{(Art. 71).}$$

When a splice is subjected to bending as well as to direct stress, the procedure must be developed by "cut and try" methods, — that is, a trial number of rivets is selected, their arrangement is established, and the stresses are investigated and compared with the allowable to note whether the rivets assumed are overstressed or understressed. If the actual stress is reasonably near and below the allowable, the joint is satisfactory; if otherwise, the number of rivets is increased or decreased accordingly. The problem of how many rivets to assume for trial is a matter of judgment and experience. As in splices for direct stresses only, it is assumed that each rivet in the group receives its proportionate part of the direct load. In other words, the direct stress on a rivet is equal to the load divided by the number of the rivets, and it is assumed to act opposite to the direction of the load. The number of rivets to assume in any given case certainly will not be less than that required to carry

the direct load. The number to add to the latter depends upon the magnitude of the moment and the relation of the rivets to each other. A rule of thumb which may be used as a guide, is to allow one extra rivet for each 50,000″# of bending.

The indirect stress on a rivet induced by the moment is assumed to act in a direction perpendicular to that of the bending, and its amount is assumed to be proportional to the distance of the rivet from the "neutral axis" of the group, similar to the general theory of beams. If r_0 represents the stress on a rivet 1″ away from such an axis, then the stress on a rivet y inches away is $r_0 \cdot y$.* The moment of this stress about the axis is $(r_0 \cdot y) \cdot y = r_0 \cdot y^2$. The moment of resistance of the group of rivets, M_r, is evidently the summation of the moments of resistance of each of the rivets, m_r, or in algebraic terms,

$$M_r = \Sigma m_r = \Sigma r_0 \cdot y^2 = r_0 \cdot \Sigma y^2.$$

The external moment, in this case M_W, must be balanced by the internal resisting moment, M_r, as in all cases of equilibrium, or $M_W = M_r$. Hence

$$M_W = r_0 \cdot \Sigma y^2, \qquad \text{or} \qquad r_0 = \frac{M_W}{\Sigma y^2},$$

which is the stress on a rivet 1″ from the axis. The stress on a rivet y inches away then is

$$r = r_0 \cdot y = \frac{M_W \cdot y}{\Sigma y^2}. \tag{1}$$

It is obvious that the farther a rivet is from the axis, the greater its indirect stress. The rivet which is the farthest away from the axis is then stressed the greatest and it controls the design. Only the extreme rivets may be stressed to their maximum allowable, and the others are stressed only to a lesser value, the amounts being proportional to the distances from the axis. If d_n represents the distance to the extreme rivet, then formula (1) becomes

$$r_k = \frac{M_W \cdot d_n}{\Sigma y^2}. \tag{2}$$

In this formula, the expression Σy^2 is often called the polar moment of inertia of the group of rivets, and it will be designated by I_r. It may be defined as the summation of the squares of the distances of all of the rivets one side of the splice line from the axis. Formula (2) may then be written as

$$r_k = \frac{M_W \cdot d_n}{I_r}. \tag{S–35}$$

This is similar to the general flexure formula

$$s = \frac{M \cdot c}{I},$$

in that r_k corresponds to s, I_r to I, and d_n to c. The resistance of a group of rivets may be imagined to be as shown in Fig. 121, if the external moment, M_e, is replaced by $P \cdot a$, an equivalent moment. The moments of the two couples must balance. As $R = P$,

$$P \cdot a = H \cdot b.$$

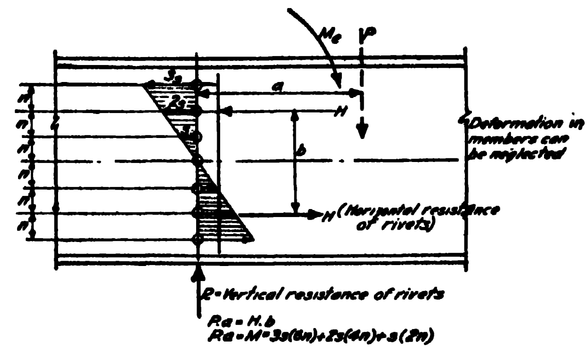


FIG. 121

In a joint which has a large number of rivets, the analysis above may not agree exactly with actual tests, but the error is probably not greater than that when it is assumed that each rivet takes an equal share of a direct load. Formula (S–35) may therefore be used for all practical purposes.

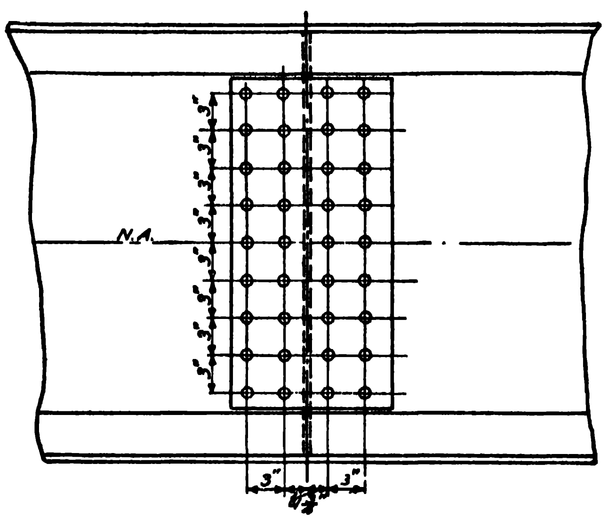


FIG. 122

Illustrative Prob. 73a. Calculate the value of I_r for the group of rivets shown in Fig. 122.

$(3)^2 = 9$
$(6)^2 = 36$
$(9)^2 = 81$
$(12)^2 = 144$
270 for 1 line above

$270 \times 2 = 540$ for 1 line
$\times\ 2$
1080 for 2 lines

$I_r = 1080.$

* This stress is assumed to vary directly as the distance from the neutral plane, — the same as in the general theory of beams.

The direct stress on each rivet may be expressed by

$r_v = \dfrac{V}{n_r}$, in which

V = the direct force (in this case the shear), and

n_r = the number of rivets on one side of the splice line.

The resultant stress may be obtained graphically as shown in Fig. 123, or analytically by

$$r = \sqrt{(r_h)^2 + (r_v)^2}. \qquad (S\text{-}36)$$

The value of r must not exceed the allowable, R, as determined by the size of the rivets used and the thicknesses of the metal involved (Art. 28).

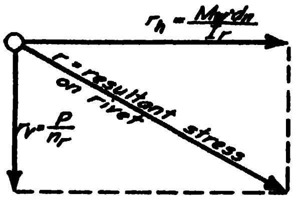


Fig. 123

The splice plates also must not become overstressed by the moment carried. The design of the plate girder section is based on an allowable stress of 16,000#/□″. In order not to have this stress exceeded, the stress at the extreme fibre of the splice plate must not exceed a value of

$$16{,}000 \times \frac{\text{the depth of the splice plate}}{\text{the effective depth of the girder}}.$$

This truth is illustrated in Fig. 124, for it is known that the stress due to bending varies directly as the

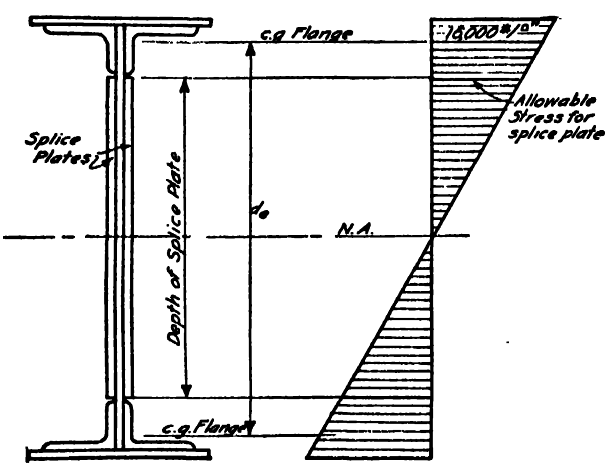


Fig. 124

distance of the fibre from the neutral axis. Employing $M = \dfrac{s \cdot I}{c}$ again, the thickness of the splice plates required for the moment may be found. The value of s must be that obtained in a manner outlined above. Thus

$$M_W = \frac{s \cdot t \cdot d^2}{6},$$

from which
$$t = \frac{6\,M_W}{s \cdot d^2}. \qquad (S\text{-}37)$$

In other words, the I of the splice plates must equal that of the web plate, both referred to the neutral axis.

The design of such a splice is largely a matter of "cut and try." To eliminate some of the work, the following method is suggested. Divide the shear value of the web by the controlling value of the rivets, R. This will be the number required for shear only and will serve as a guide. If the controlling value of the rivet, R, is divided by one-half the effective depth of the girder, $\frac{1}{2}\,d_e$, the value of a rivet 1″ from the neutral axis in resisting moment will result. Arrange one line of splice rivets in a tentative way for trial, and calculate the value of Σy^2 for this line. M_W divided by Σy^2 and the value for a rivet 1″ from the neutral axis, will give the number of lines of rivets theoretically required. This result will be a number with decimals. If the next whole number above is taken as the number of lines, there will probably be enough rivets to carry the shear also. The stress on the outermost rivet can now be calculated by the formula for this arrangement and compared with the allowable. An advantage in obtaining this trial number of lines is that the designer is given an opportunity to decide whether a splice of the type (a) in Fig. 120 may be used, or whether a splice of the type (c) or (d) will have to be employed. If the number of trial lines exceeds three, the latter types should be used. In such a case, the number of rows in the vertical splice plate may be limited to two either side of the splice line. The moment of resistance of the rivets in this plate may be calculated and subtracted from M_W. The resistance of a trial line in the horizontal splice plate may then be calculated and this value divided into the remaining value of M_W to obtain the number of trial lines required. As before, with an arrangement determined, the stress on the outermost rivet may be calculated and checked with the allowable.

Illustrative Prob. 73b. Design a splice for a $42 \times \frac{5}{16}$ web plate. $\frac{1}{8}\,A_W$ used for moment resistance. $6 \times 6 \times \frac{3}{8}$ flange angles (net area, two holes out of each angle, = 8.80□″), 2-14 × $\frac{3}{8}$ cover plates (net area, two holes out of each plate, = 9.18□″). L = 40′-0″. w = 5000#/ft. d_e = 41.12″.

$$M = 1.5\,w \cdot L^2 = 1.5 \times 5000 \times (40)^2 = 12{,}000{,}000''\#$$

$$\tfrac{1}{8}\,A_W = \tfrac{1}{8} \times 42 \times \tfrac{5}{16} = 1.64\square''$$

$$A_{FN} = 1.64 + 8.80 + 9.18 = 19.62\square''$$

$$M_W = 12{,}000{,}000 \times \frac{1.64}{19.62} = 1{,}000{,}000''\#$$

$$V_{max} = 42 \times \tfrac{5}{16} \times \tfrac{3}{4} \times 12{,}000 = 118{,}000\#$$

b. to b. Ls = $42 + \frac{1}{2} = 42\frac{1}{2}''$. $42\frac{1}{2} - 2 \times 6 - 2 \times \frac{1}{4} = 30''$.

Use 30″ Splice Plates.

$$118{,}000 = 12{,}000 \times 30 \times \tfrac{3}{4} \times 2\,t \qquad t = 0.22'' \text{ for shear.}$$

Maximum allowable stress at edge of splice plate

$$16{,}000 \times \frac{30}{41.12} = 11{,}660\#/\square''$$

$$1{,}000{,}000 = \frac{11{,}660 \times 2\,t \times (30)^2}{6} \qquad t = 0.286'' \text{ for moment.}$$

Use 2-$\frac{5}{16}$" plates.

Try vertical plates only

10 spaces @ $2\frac{3}{4}'' + 2 \times 1\frac{1}{4}''$ edge distance = 30"

$(2.75)^2 = 7.5$ — R for $\frac{3}{4}''$ rivet = 5630
$(5.50)^2 = 30.25$ — R for 1" from N.A. $= \frac{5630}{20.56} = 274\#$
$(8.25)^2 = 68$
$(11.00)^2 = 121$
$(13.75)^2 = 188$
Total: 415
× 2
830 Resistance of 1 line $274 \times 830 = 247{,}000''\#$

$$\frac{1{,}000{,}000}{247{,}000} = 3+ \quad \text{Must use horizontal plates}$$

Use upper two rows in horizontal plate.

$(2.75)^2 = 7.5$ — Use 2 lines in vertical plate
$(5.50)^2 = 30.5$ — $2 \times 212 = 424$
$(8.25)^2 = 68$
106 — $424 \times 274 = 116{,}000''\# = M_r$ of vertical plate
× 2
212 (1 line)

$1{,}000{,}000 - 116{,}000 = 884{,}000''\#$ by horizontal plates

$(11)^2 = 121$ — $618 \times 274 = 169{,}300''\# = M_r$ of 1 line in horizontal plate.
$(13.75)^2 = 188$
309
× 2
618 (1 line)

$$\frac{884{,}000}{169{,}300} = 5+. \quad \text{Try 5 lines in horizontal plate.}$$

I_r of arrangement $= 424 + 5 \times 618 = 3514$

$d_n = 13.75$

$$r_h = \frac{M_W \cdot d_n}{I_r} = \frac{1{,}000{,}000 \times 13.75}{3514} = 3930\#$$

34 rivets one side of splice line

$$- \frac{118{,}000}{34} = 3480\#$$

$$\left.\begin{array}{ll} r = \sqrt{(3480)^2 + (3930)^2} & = 5340\# \\ r \text{ (allowable)} & = 5630 \end{array}\right\} \text{O.K.}$$

Prob. 73c. Design a splice for a $48 \times \frac{3}{8}$ web plate. $\frac{1}{8}$ A_W used for moment resistance. $6 \times 6 \times \frac{1}{2}$ flange angles (net area, two holes out), 2-14 × ½ cover plates (net area, two holes out, each plate). $M_{max} = 20{,}000{,}000''\#$. Calculate d_e.

74. Cover Plate Splices.

Quite frequently required lengths of cover plates are greater than those available (Art. 8) and it becomes necessary to splice them. Cover plates should be spliced with plates of equal section, with the same dimensions, as illustrated in Fig. 125 (*a*).* The **exact method** for determining the pitch of rivets required is to calculate the amount of horizontal shear per linear inch at the plane where the cover plate under consideration rests. The following formula is again used:

$$q \cdot b = \frac{V \cdot Q}{I}.$$

If the value of this shear ($q \cdot b$) is x pounds, then the spacing is determined by dividing the controlling value of the rivet (single shear or bearing) by this value; or

$$p = \frac{R}{x} \text{ (as before).}$$

An **approximate solution** is more common in practice. The number of rivets used in the splice should develop the full strength of the plates to be spliced. This results from dividing the product of the net area of the plate and 16,000#/□" by the controlling value of the rivet, or

$$n_r = \frac{A_{CN} \times 16{,}000}{R}. \qquad (S\text{-}38)$$

When the number of rivets is known, they may be arranged in accordance with the details as determined by the point of splice. When the splice

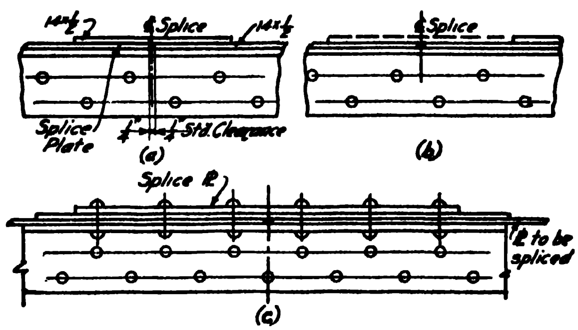


FIG. 125

plate is not in direct contact with the plate to be spliced, as shown in Fig. 125 (*c*), the rivets are not as efficient in transferring stress and some allowance is usually made for this condition. A common specification is to require one-third more rivets on each side of the splice line than those theoretically required, for each intervening plate. Due to disadvantages of long rivets through intermediate plates being subjected to bending, poor appearance, uneven surfaces to start walls upon, and so on, cover plate splices are unsatisfactory, and they should be avoided when possible. For these reasons it is quite common practice to splice one cover plate with another. The top cover plate may usually be obtained in one length, and it may be used to splice the other plates. Ingenuity may be exercised in the

* Some designers plan to butt the ends of the cover plates in the compression flange at the splice line instead of allowing the clearance shown for the tension flange.

selection of the splice location by making the splice line where the plate above would otherwise stop, and running the latter plate far enough beyond the splice line to accommodate the splice rivets as suggested by the dotted lines in Fig. 125 (*b*). If the plate above is thinner than the plate to be spliced, the former may be carried over to a point where a cover plate of a thickness equal to the thinner plate is required only. The above reasoning is based on the assumption that the stress in the end of the plate to be spliced is transferred to the plate just above it and that beyond this point it takes its increment of stress as usual.

Illustrative Prob. 74a. Design the splice required for a $20 \times \frac{7}{8}$ cover plate (2 holes out, $\frac{7}{8}''$ rivets).

Net area $20 \times \frac{7}{8} = 15.75\square''$
$15.75 \times 16{,}000 = 252{,}000\#$ maximum tension
Single shear $\frac{7}{8}''$ rivet $= 7220\#$

$\frac{252{,}000}{7220} = 35+$ Use 36 rivets, 18 each side of center line of web. Space in two lines staggered, $3\frac{1}{2}''$ o.c.

Use $20 \times \frac{7}{8}$ splice plate.
Length $= 10'$-$6''$

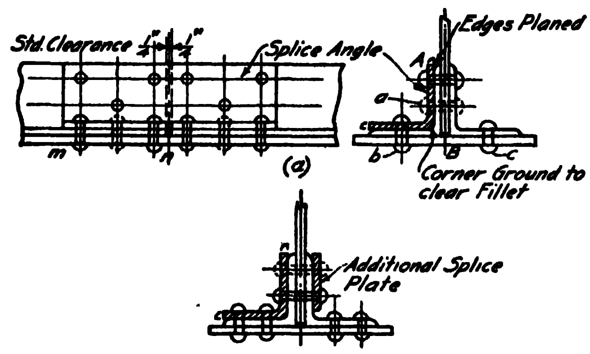
If a plate intervened between the splice plate and the plate spliced, $\frac{1}{3}$ would be added to the number of rivets theoretically required, or

$1.33 \times 36 = 48$ 48 rivets would have to be used.

Prob. 74b. Design a splice for a $14 \times \frac{1}{2}$ cover plate (2 holes out, $\frac{3}{4}''$ rivets).

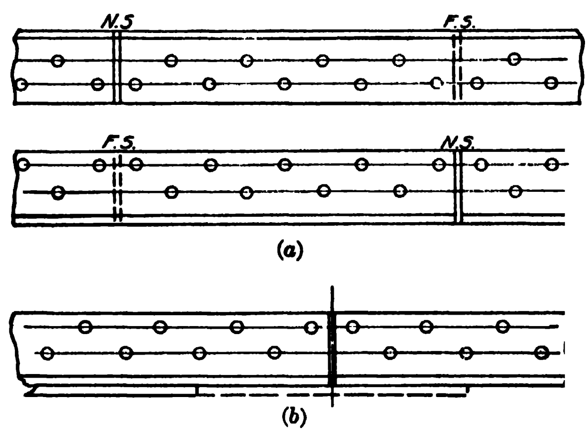
75. Flange Angle Splices.

Since angles can generally be obtained in longer lengths than plates, flange angle splices are not required as often as other kinds of splices. When a splice is necessary, only one angle should be spliced at a time. The usual splice is made by a length of **angle of equal net section** riveted to both legs of the flange angle, as in Fig. 126 (*a*). Usually a splice angle is selected with legs of the same width as those of the flange angle. The projections are planed off as shown at *A* in the figure to make a more workmanlike job, and the corner, shown at *B*, is also ground off to clear the fillet of the flange angle. Special gauges in the angles will be required in order to drive the rivets properly. An angle of the same size as the flange angle but of greater thickness will be required, inasmuch as some of the area has been eliminated by grinding or milling. If the required net section cannot be supplied by one angle without exceeding the maximum thickness allowable (equal to the diameter of the rivet usually), the largest possible splice angle is used and the remainder of the required area is made up by a plate riveted to the vertical leg of the flange angle on the far side, as in Fig. 126 (*b*). In order to keep splices sufficiently staggered, the top angle should be spliced on the near side and the bottom angle on the far side, and the reverse order should be followed on the other side of the center line of the span. This is diagrammatically illustrated in Fig. 127 (*a*) and it will keep the punching symmetrical. The splice for the compression flange angles is made the same as that for the tension flange angles, for reasons of symmetry.



(*b*)
FIG. 126

The **number of rivets** required for a splice, where no cover plate exists, can be obtained by dividing the value of the net section of the angle in tension by the controlling value of the rivet (usually single shear). It is unnecessary to allow for the variation of the flange stress due to the changing moment here, for the rivets are able to carry the full value of the angle and therefore can carry any increment



(*b*)
FIG. 127

of stress. When cover plates occur at the flange splice, the number of splice rivets calculated as above should be increased to cover the increase of flange stress occurring in the length of the splice angle. The extra number required cannot be accurately obtained unless the length of the splice

angle is known in advance.* Since the splice usually occurs where the increment is small, it is generally sufficient to allow two or three extra rivets for this purpose.

The required number of rivets should be proportioned between the legs of the splice angle according to their areas. If the legs are equal, one-half of the rivets are placed in each leg, and so on. Rivets already transferring horizontal shear may be considered as splice rivets if they are included in the splice angle. The rivets should be placed close together to make the transfer in as short a distance as possible, but the net section of the basic design must be maintained. As before, a cover plate carried beyond where it is needed will aid the splice, as indicated by the dotted lines in Fig. 127 (*b*).

Illustrative Prob. 75a. Design a splice for a 6 × 6 × ½ flange angle (2 holes out, ¾" rivets).

Gross section = 5.75□"
2 holes ⅞ × ½ = 0.87
4.88□"

Tensile strength = 4.88 × 16,000 = 78,000#
Single shear ¾" rivet = 5300#
Bearing on ½" metal = 9000#

$\frac{78,000}{5300} = 14+$ Use 16 rivets, 8 in each leg. Space 3" o.c. staggered.

Splice ∟ 5¼ × 5¼ (Planed from 6 × 6)
5¼ × 5¼ × ¾ = 7.69
2 holes ⅞ × ¾ = 1.31
6.38□"
5¼ × 5¼ × ⅝ = 6.48
2 holes ⅞ × ⅝ = 1.09
5.39

Use 5¼ × 5¼ × ⅝ × 4'-0" Splice angle.

Prob. 75b. Design a splice for an 8 × 8 × ¾ flange angle (3 holes out, ⅞" rivets).

SECTION 5I

DESIGN SUMMARY

76. Typical Design Example.

In order to review the principles discussed in the preceding articles, the following problem is given. The reasons and article references are often shown. It is understood that such explanatory notes are not a part of the design. A typical design sheet would be carried out more in the order of Art. 39.

Design the plate girder for the condition of loading shown in Fig. 128 (*a*). Height from the finished floor to the bottom of the girder fire protection must be approximately 8'-0" on account of door openings, and so on.

The span will be taken as 59'-0" (assuming the *H* columns to be 12" as a minimum), as in Art. 40. An approximation for the weight of the girder must be made. From Art. 42,

$$w_0 = 3 + \frac{L}{4.25} = 3 + \frac{59}{4.25} = 16.6 \text{ say } 17\#/\square'.$$

L.L. = 200
1" Grano. Fin. = 12
10" Slab = 120
Plastered Ceiling Direct = 5
Girder = 17
T.L. = 354 say 355#/□'.

The approximate height of the partition above, as in (*b*), is 14'-0" − (1'-4") = 12.67'. 6" T.C. plastered two sides = 35# per superficial foot. The cross section shown in Fig. 128 (*c*) will be used in an effort to reduce the weight of the girder fire protection. 3" T.C. plastered one side = 22# per superficial foot. Its height is approximately 4'-7". Assuming 14" for the dimension *a*, the width of concrete is 18". Assuming 3" as the thickness of the total flange metal, the thickness of concrete will be approximately 8". The load per foot may now be calculated.

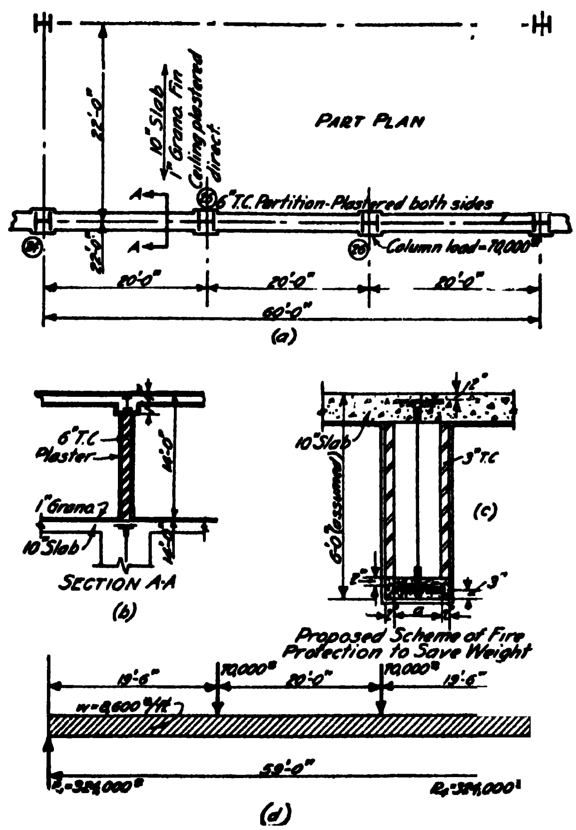


FIG. 128

* Referring to Fig. 126 (*a*), Prof. C. M. Spofford in his "Theory of Structures" states: ". . . the rivets at *a* must carry, from the flange into the splice angle in the distance *mn*, one-half the increment in flange stress in that distance (the other half going through the same rivet to the flange angle on the left-hand side) plus one-half the stress in the main angle at *m* (since the angle is equal-legged). The rivets at *c* should be computed to carry the same amount, since it is proper to assume that all the increment in flange stress is carried by the cover plates, the angle being fully stressed before cover plates are added."

Load per linear foot.

Floor	= 355 × 22	= 7810
Partition	= 12.67 × 35	= 444
F.P. T.C.	= 2 × 4.58 × 22	= 201
Concrete	= 18 × 8	= 144
		8599 say 8600#/ft.

The load from the columns #25 and #26 is 70,000# each, obtained from other data. Figure 128 (*d*) shows the loading diagram.

$$R_1 = R_2 = \frac{59 \times 8600}{2} + 70{,}000 = 323{,}700 \text{ say } 324{,}000\#.$$

The maximum moment occurs at the center of span.

$$M_{\max} = 324{,}000 \times 29.5 - \frac{8600 \times (29.5)^2}{2} - 70{,}000 \times 10$$
$$= 5{,}110{,}000'\#.$$

Web Plate.

$V = 324{,}000\#$. Allowable $v = 12{,}000\#/\square''$ (Art. 45).

$$A_{WN} = \frac{V}{v} = \frac{324{,}000}{12{,}000} = 27\square'' \text{ net area of web plate required.}$$

$A_{WN} = \frac{3}{4} A_W$, the gross area (Art. 45).

Hence $\frac{4}{3} \times 27 = 36\square''$ gross area of web plate required.

The depth of girder should be between $\frac{L}{10}$ and $\frac{L}{12}$ (Art. 41).

$\frac{59}{10} = 5.9' = 71''$ $\frac{59}{12} \times 12 = 59''$ Try 66″ Web Plate on account of limiting conditions (see statement of problem). That is, 14′-0″ − (8′-0″) = 6′-0″ = 72″. Allowing for fire protection, 66″ maximum available.

$$\frac{A_W}{d_W} = \frac{36}{66} = 0.54 \qquad \text{Use } 66 \times \tfrac{9}{16} \text{ Web Plate.}$$

$$\text{O.K.} > \frac{66}{160} = 0.41'' \text{ (Specification Art. 45).}$$

Flanges.

$M_{\max} = 5{,}110{,}000'\# = 61{,}320{,}000''\#$

The trial effective depth will be taken as the depth of web plate.

$$\text{Flange stress, } F = \frac{61{,}320{,}000}{66} = 933{,}000\# \text{ (Art. 47)}$$

$$\text{Total flange area required (net)} = \frac{933{,}000}{16{,}000} = 58.2\square''$$

$\frac{1}{8}$ the gross area of the web will be allowed as available flange material. $\frac{1}{8} A_W = \frac{1}{8} \times 66 \times \frac{9}{16} = 4.64\square''$. The amount to be supplied by the flange proper is then 58.2 − 4.64 = 53.56□″. As a guide, about one-half the flange area should be in the flange angles if possible (Art. 37).

$$\frac{53.56}{2} = 26.78\square'' \qquad \frac{26.78}{2} = 13.39\square'' \text{ for one angle (net).}$$

$\frac{7}{8}''$ rivets are to be used. The largest available angle is then 8 × 8 × $\frac{7}{8}$, unless 1″ rivets are used.

Gross area 1 − 8 × 8 × $\frac{7}{8}$ ∟ = 13.23□″
2 holes out each ∟ = 2 × 1 × $\frac{7}{8}$ = 1.75
11.48□″ net area—1 angle.

53.56 − (2 × 11.48) = 30.6□″ to be supplied by the cover plates. With 2-8 × 8 angles, 18″ or 20″ cover plates are used (Art. 49).

Try 2–18 × $\frac{3}{4}$ Pls. and 1–18 × $\frac{5}{8}$ Pl.*

Gross area = 18 × 0.75 = 13.50□″
2 holes out = 2 × 1 × $\frac{3}{4}$ = 1.50
Net area = 12.00□″

* 2-18 × $\frac{7}{8}$ C.Pls. were tried and found insufficient. Also 2-20 × $\frac{7}{8}$ Pls. are insufficient. If 3 plates are necessary, an 18″ width may as well be used as a 20″ width.

30.6 − (2 × 12.00) = 6.6□″

Gross area 18 × $\frac{5}{8}$ Pl. = 18 × 0.62 = 11.25□″
2 holes out = 2 × 1 × $\frac{5}{8}$ = 1.25
10.00□″

Some excess of flange area should be provided in addition to that theoretically required to allow for contingencies. Hence the following make-up of flange material will be used:

$\frac{1}{8} A_W$ = 4.64 = $\frac{1}{8}$ of 66 × $\frac{9}{16}$ Web Pl.

2∟s (2 holes out each angle) = 22.96 = 2-8 × 8 × $\frac{7}{8}$ flange angles

2 Cover Plates (2 holes out each plate) = 24.00 = 2-18 × $\frac{3}{4}$ Cover Plates

1 Cover Plate (2 holes out) = 10.00 = 1-18 × $\frac{5}{8}$ Cover Plate

61.60□″ Total net area }
58.20□″ Required } O.K.

The section should now be tested to note whether the distance between the centers of gravity of the flanges is within the distance back to back of angles (Art. 37). Referring to Fig. 129,

$$\frac{2 \times 11.48 \times 30.93 + 12.0 \times 33.62 + 12.0 \times 34.37 + 10 \times 35.06}{2 \times 11.48 + 12.0 + 12.0 + 10.0} = 33''$$

Distance c.g. Flange to c.g. Flange = 2 × 33 = 66″ }
Distance back to back of Flange ∟s = 66.5″ } O.K.

The above calculation is based upon net sections. The result is not materially different if gross sections were used.

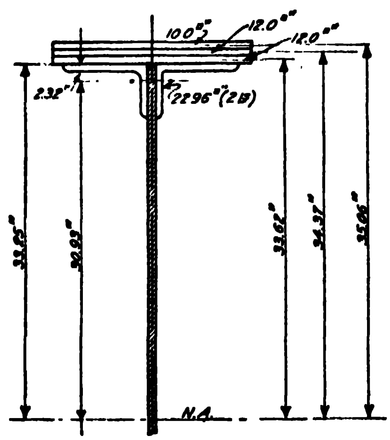


Fig. 129

The next step is to determine the **cover plate lengths.** The values of the moment at a sufficient number of points along the span must be calculated to establish the moment curve. Thus

$$M \text{ @ } 4'\text{-}0'' \text{ from } R_1 = 324{,}000 \times 4 - \frac{8600 \times (4)^2}{2}$$
$$= 1{,}227{,}200'\#$$

$$M \text{ @ } 8'\text{-}0'' \text{ from } R_1 = 324{,}000 \times 8 - \frac{8600 \times (8)^2}{2}$$
$$= 2{,}316{,}800$$

$$M \text{ @ } 12'\text{-}0'' \text{ from } R_1 = 324{,}000 \times 12 - \frac{8600 \times (12)^2}{2}$$
$$= 3{,}268{,}800$$

$$M \text{ @ } 16'\text{-}0'' \text{ from } R_1 = 324{,}000 \times 16 - \frac{8600 \times (16)^2}{2}$$
$$= 4{,}084{,}000$$

$$M \text{ @ } 19'\text{-}6'' \text{ from } R_1 = 324{,}000 \times 19.5 - \frac{8600 \times (19.5)^2}{2}$$
$$= 4{,}680{,}000$$

$$M \text{ @ } 24'\text{-}0'' \text{ from } R_1 = 324{,}000 \times 24 - \frac{8600 \times (24)^2}{2}$$
$$- 70{,}000 \times 4.5 = 4{,}984{,}200$$

$$M \text{ @ } 28'\text{-}0'' \text{ from } R_1 = 324{,}000 \times 28 - \frac{8600 \times (28)^2}{2}$$
$$- 70{,}000 \times 8.5 = 5{,}105{,}800$$

M @ C.L. = maximum, as already calculated = 5,110,000″#

These values are plotted in Fig. 130, the flange areas are superimposed upon the diagram, and the resulting cover plate lengths are found, as described in Art. 53. The results are:

1-18 × ¾ C.Pl. × 46′-0″
1-18 × ¾ C.Pl. × 36′-0″
1-18 × ⅝ C.Pl. × 26′-0″

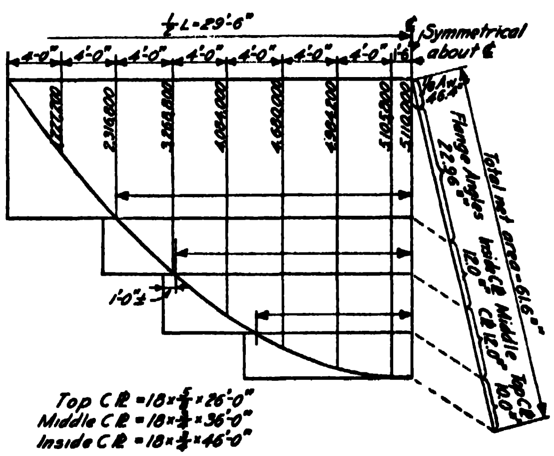


FIG. 130

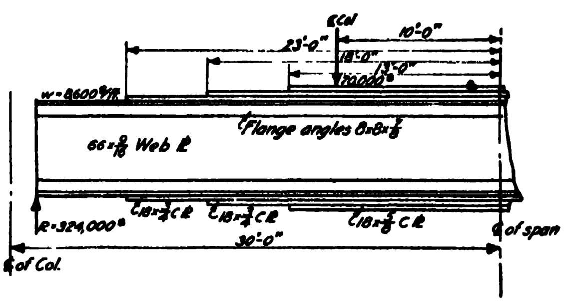


FIG. 131

The engineer's sketch (shown in Fig. 131) often includes only the information calculated thus far. The remaining computations are more in the order of detail design and often are supplied and submitted for approval by the structural fabricator.

Stiffeners @ interior concentrated load. P = 70,000#

$$\text{Bearing area required} = \frac{P}{p} = \frac{70{,}000}{20{,}000} = 3.50\square''$$

4 Ls are required under the column for practical reasons.

$$\frac{3.50}{4} = 0.87\square'' \text{ area required for one L.}$$

The size should be 6 × 3½ (Table 28).

Available length of bearing = width of outstanding leg of stiffener angle − radius of the fillet of the flange angle, or 6 − ⅝ = 5.37″ (Fig. 132).

$$t = \frac{0.87}{5.37} = 0.16'' \text{ required. The minimum available is } \tfrac{3}{8}''.$$

Hence use 4-6 × 3½ × ⅜ Ls. The number of rivets required may be calculated as follows:

Enclosed bearing on web = ⅞ × 9/16 × 30,000 = 14,780#

Double shear = 24,000 × 0.6013 = 14,430# controls

$$\text{No. rivets} = \frac{70{,}000}{14{,}430} = 4.8 \text{ say } 5.$$

The maximum allowable spacing of 6″ will control the design here however. The arrangement is shown in Fig. 132.

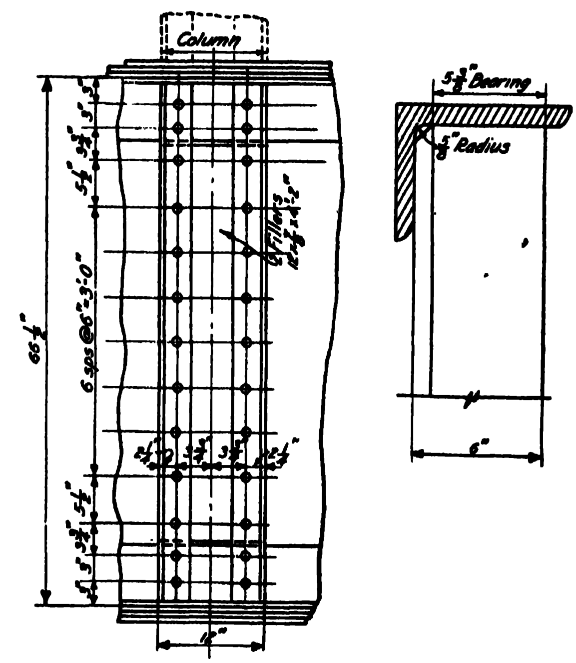


FIG. 132

Intermediate Stiffeners.

$$\text{o.s. leg not less than } \frac{1}{30}(\text{depth of web plate}) + 2'' = \frac{66}{30} + 2$$
$$= 4.2.$$

Use 5 × 3½ × 5/16 stiffener angles × 5′-4¾″ and 3 × ⅝ bar fillers × 4′-2″.

The spacing of intermediate stiffener angles may be obtained from

$$d_s = \frac{t}{40}\left(12{,}000 - \frac{V}{A_{WN}}\right) \text{ (Art. 59).}$$

$$V \text{ @ } 1'\text{-}0'' \text{ from } R_1 = 324{,}000 - 8600 = 315{,}400$$

$$d_s = \frac{0.56}{40}\left(12{,}000 - \frac{315{,}400}{27.8}\right) = 9.1'' \text{ say } 10''.$$

It will be more economical to revise the web plate size, making it 66 × ⅝. Hence

$$t = 0.63 \quad A_{WN} = \tfrac{3}{4} \times 66 \times 0.63 = 31.2\square''$$

V @ 1'-6" from $R_1 = 324{,}000 - 1.5 \times 8600 = 311{,}100$

$$d_s = \frac{0.63}{40}\left(12{,}000 - \frac{311{,}100}{31.2}\right) = 33'' \text{ say } 2'\text{-}6''$$

V @ 5'-0" from $R_1 = 324{,}000 - 5 \times 8600 = 281{,}000$

$$d_s = \frac{0.63}{40}\left(12{,}000 - \frac{281{,}000}{31.2}\right) = 48.7'' \text{ say } 4'\text{-}0''$$

V @ 11'-0" from $R_1 = 324{,}000 - 11 \times 8600 = 229{,}400$

$$d_s = \frac{0.63}{40}\left(12{,}000 - \frac{229{,}400}{31.2}\right) = 74'' \text{ use } 5'\text{-}0''.$$

Specifications require that the maximum spacing of intermediate stiffeners shall not exceed the depth of the girder, or in any case 5'-0". The above calculations are used as a guide, and the stiffeners must be arranged to meet actual conditions as shown in Fig. 133.

This method is too laborious to be of use commercially and an approximation may be made by employing

$$p = \frac{R \cdot d_e}{V} \times \frac{A_{FN} + \frac{1}{8}A_W}{A_{FN}}. \quad \text{(Art. 60).}$$

$$p = \frac{14{,}430 \times 61.86}{302{,}500} \times \frac{27.6}{22.96} = 3.54''.$$

A still simpler solution may be made by using

$$p = \frac{R \cdot d_e}{V}$$

in which the effective depth, d_e, is taken as $\frac{1}{2}''$ less than the depth of the web plate.

$$p = \frac{14{,}430 \times 65.5}{302{,}500} = 3.07'' \qquad \text{Use } 3''$$

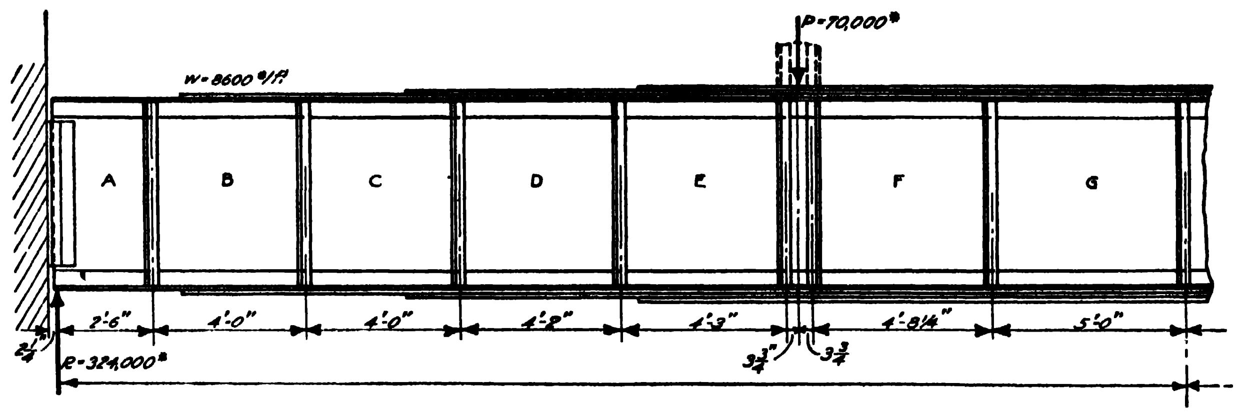


Fig. 133

Spacing of Flange Rivets. The exact method involves calculating the horizontal shear per linear inch. For panel A, Fig. 133, the angles constitute the flange. Referring to Fig. 129.

The statical moment $Q = A \cdot d = 26.46 \times 30.93 = 820''^3$

The shear per linear inch $= bq = \dfrac{VQ}{I}$

I (the inertia)

$$66 \times \tfrac{9}{16} \text{ Pl.} = \frac{bd^3}{12} = \frac{0.5625 \times (66)^3}{12} = 13{,}470$$

$$I \text{ of } 4 \text{ Ls} = 4 \times 79.6 = 320$$

$$Ad^2 \text{ of Ls} = 13.23 \times 30.93^2 \times 4 = 50{,}600$$

$$64{,}390''^4 \text{ Total}$$

V @ C.L. panel $A = 324{,}000 - 8600 \times 2.5 = 302{,}500$

$$bq = \frac{302{,}500 \times 820}{64{,}390} = 3950\#/\text{in.}$$

The load directly on the top flange $= \dfrac{8600}{12} = 720\#/\text{in.}$

The resultant $= \sqrt{(3950)^2 + (720)^2} = 4140\#$

The controlling value of the rivet = 14,430# (see preceding computation).

$$p = \frac{14{,}430}{4140} = 3.48'' \text{ pitch of rivets in panel } A.$$

The remaining values of the pitch figured in this manner are as follows:

Panel B.	$p = \dfrac{14{,}430 \times 65.5}{284{,}500} = 3.33''$	Use $3\frac{1}{4}''$
C.	$p = \dfrac{14{,}430 \times 65.5}{244{,}800} = 3.82''$	Use $3\frac{3}{4}''$
D.	$p = \dfrac{14{,}430 \times 65.5}{201{,}900} = 4.68''$	Use $4\frac{1}{2}''$
E.	$p = \dfrac{14{,}430 \times 65.5}{158{,}900} = 5.95''$	Use $5\frac{3}{4}''$
F.	$p = \dfrac{14{,}430 \times 65.5}{115{,}900} = 8.17''$	Use 6"

When the calculated value exceeds 6", no further calculations are necessary, as the rivets must be spaced 6" on centers as a maximum. The above method is one which is often used in commercial design rather than some other more involved method. It will be noted that when the actual pitches to be used are selected (to the nearest $\frac{1}{4}''$ below), the same selection results in many cases for a given panel, irrespective of the method used. The influence of the direct load at the top chord on the rivet pitch is small in this case, as illustrated above, and therefore it is neglected in the remainder of the calculations.

Cover Plate Rivets. The largest horizontal shear affecting the cover plates occurs at the end of the inside plate. The exact method involves the shear per linear inch as before.

Q (the statical moment) of the cover plate about the

neutral axis = 18 × ¾ × 33.62 = 463″³. The moment of inertia of the whole cross section with one cover plate top and bottom (see Fig. 129) is

$$I \text{ of } 66 \times \tfrac{9}{16} \text{ Pl.} = \frac{b \cdot d^3}{12} = \frac{0.5625 \times (66)^3}{12} = 13{,}470$$

$$I \text{ of } 4 \text{ Ls} = 4 \times 79.6 = 320$$

$$A \cdot d^2 \text{ for Ls} = 13.23 \times 4 \times \overline{30.93}^2 = 50{,}600$$

(I of cover plate about own axis neglected)

$$A \cdot d^2 \text{ for C.Pl.} = 18 \times \tfrac{3}{4} \times \overline{33.62}^2 \times 2 = \underline{31{,}200}$$

$$95{,}590''^4 \text{ (Total)}$$

V @ end of cover plate = V @ 6′-6″ from R_1 = 324,000 − 8600 × 6.5.

$$V = 268{,}100\#$$

$$q \cdot b = \frac{268{,}100 \times 463}{95{,}590} = 1300\#/\text{in.}$$

The rivets are in single shear, and the controlling value = 7220#. There are 2 rivets in a transverse line. Hence

$$p = \frac{2 \times 7220}{1300} = 11.1''.$$

Such a solution is not used in practice and an approximate method is sufficiently accurate. Thus

$$p = n \cdot \frac{R \cdot d_e}{V} \cdot \frac{A_{FN}}{A_{CN}} \text{ (Art. 64).}$$

The effective depth may be assumed as ½″ less than the depth of web plate as before. The net section where one cover plate occurs is 4.64 + 22.96 + 12.00 = 39.6□″ = A_{FT} at this point. Then

$$p = \frac{2 \times 7220 \times 65.5 \times 39.6}{268{,}100 \times 12.0} = 11.6''.$$

This value agrees well with the exact method. The maximum allowable pitch is 6″ so that this determines the pitch for the cover plates. This is usually the case so that neither the approximate nor the exact calculations would influence the pitch to be used generally. It is wise to make the calculation for the extreme case to show that a 6″ pitch is satisfactory, however. Many designers believe that there should be enough rivets at close spacing at the end of a cover plate to develop its strength. Thus

A_{CN} = 12.00□″ Its tensile strength = 12.00 × 16,000 = 192,000#

$$\frac{192{,}000}{7220} = 27 \text{ rivets.}$$

Hence use 28 rivets at the ends of the cover plates at 3″ alternate spaces. All cover plate rivets should be checked in location to see that they do not "foul" any stiffeners.

Web Splice. Since the length of the girder is 59′-0″, one web splice is required if the maximum available length of plate in this case is 33′-0″. Detail dimensions:

¾″ from b. to b. of Ls at each end.
59′-0″ − (2 × ¾) = 58′-10½″
58′-10½″ − (2 × ¼″ clearance at splice) = 58′-10″
58′-10″ − (33′-0″) = 25′-10″
Use 66 × 9/16 × 33′-0″
66 × 9/16 × 25′-10″.

C.L. of splice located 25′-11″ from R_1. The web must be spliced for capacity.

A_{WN} = 66 × 9/16 × ¾ = 27.9□″. V_{max} = 27.9 × 12,000 = 335,000#.

Unenclosed Bearing ⅞″ φ rivet

24,000 × ⅞ × 9/16 = 11,800# controls
Double Shear = 14,430#

$$\text{No. of rivets required for shear} = \frac{335{,}000}{11{,}800} = 29$$

⅛ A_W has been counted upon as flange material. Therefore, the splice must carry this proportion of the moment.

$$M_{max} = A_{FT} \times d_e \times 16{,}000 = 61.6 \times 66 \times 16{,}000 = 65{,}000{,}000''\#$$

$$M_W \text{ (the moment carried by the web)} = M_{max}\,\frac{\tfrac{1}{8}A_W}{A_{FT}}$$

$$M_W = 65{,}000{,}000 \times \frac{4.64}{61.6} = 4{,}900{,}000''\#.$$

A trial arrangement must be assumed. Considering three rivets in a vertical line in the horizontal plate in Fig. 134, the polar moment of inertia is

$$\overline{23.5}^2 = 552.5$$
$$\overline{20.5}^2 = 420$$
$$\overline{17.5}^2 = \underline{306}$$
$$1278.5 = 2\Sigma d^2 \text{ for these 3 rivets.}$$

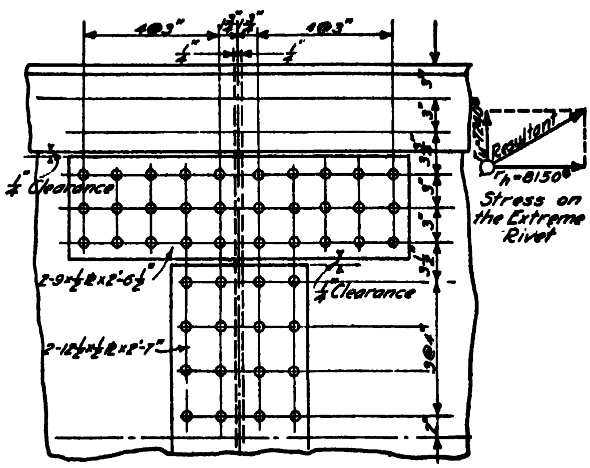


FIG. 134

The same value occurs for the 3 rivets in the horizontal plate at the bottom. Thus

$$2 \times 1278.5 = 2557 = \Sigma d^2 \text{ for rivets in one line}$$

(neglecting the rivets in the vertical plate for the time being). The value of a rivet 1″ from the N.A. is

$$\frac{11{,}800}{33} = 358\#.$$

The moment of resistance of the rivets in one vertical line of the two horizontal plates is

$$358 \times 2557 = 915{,}000''\#$$

$$\text{The number of lines required} = \frac{4{,}900{,}000}{915{,}000} = 5.3.$$

Try 5 lines. The arrangement is shown in Fig. 134.

Σd^2 for vertical plate

$\overline{2}^2 = 4$	336 × 2 = 672 for 1 vertical line
$\overline{6}^2 = 36$	672 × 2 = 1344 for vertical Pl.
$\overline{10}^2 = 100$	
$\overline{14}^2 = \underline{196}$	
336 (above N.A.)	

Σd^2 for horizontal Pls.

$$
\begin{array}{rrl}
\overline{23.5}^2 = & 552.5 & \\
\overline{20.5}^2 = & 420 & \\
\overline{17.5}^2 = & \underline{306} & \\
& 1278.5 & \text{for 1 line in 1 Pl.} \\
& \underline{2} & \\
& 2557 & \text{for 1 line in Pls. top and bottom lines} \\
& \underline{5} & \\
& 12{,}785 & \text{for vertical Pls.} \\
& \underline{1{,}344} & \text{for horizontal Pl.} \\
& 14{,}129 & \text{total.}
\end{array}
$$

$$r_h = \frac{M_W \times d_n}{I_r} \text{ (Art. 73)}$$

$$r_h = \frac{4{,}900{,}000 \times 23.5}{14{,}129} = 8150\#$$

$$r_v = \frac{V}{n} = \frac{333{,}000}{46} = 7240\#$$

$$\left.\begin{array}{r}\text{Resultant} = \sqrt{(8150)^2 + (7240)^2} = 10{,}900\# \\ R \text{ (allowable)} = 11{,}800\#\end{array}\right\}\text{O.K.}$$

The above design may be altered somewhat if desired so that the actual stress on the extreme rivet is nearer the allowable by using a few less rivets, but this splice will be used.* It is always advisable to be conservative in the design of splices. The thickness of splice plates required will either be controlled by the shear or by the moment, carried by the web.

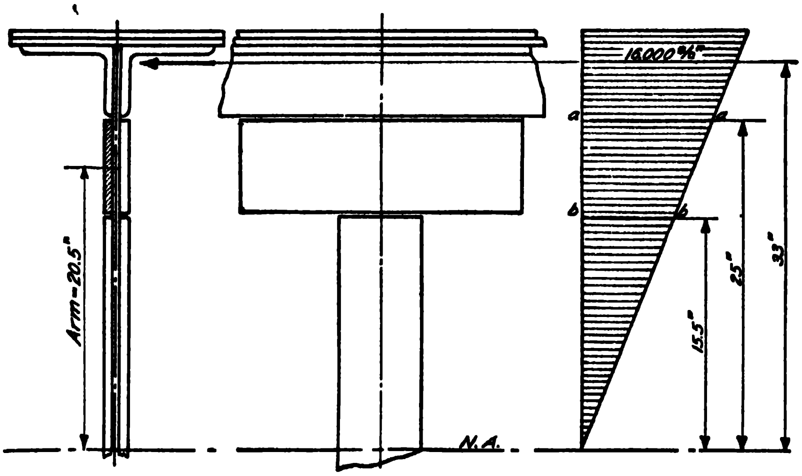


FIG. 135

A_{WN} from above = 27.9□". Height of the three plates = 9 + 9 + 31 = 49".

$$\frac{27.9}{49} = 0.57''. \quad \tfrac{5}{16}'' \text{ Pl. each side for shear.}$$

To safely resist the bending, the stress at the edge of the outside splice plate must be such that the stress at the center of gravity of the flange does not exceed 16,000#/□" as already designed. This stress may be found by the proportionate distances to the respective fibres. Thus in Fig. 135,

$$\frac{\text{Stress @ } a\text{–}a}{16{,}000} = \frac{25}{33} \qquad \text{Stress @ } a\text{–}a = 12{,}120\#/\square''$$

$$\frac{\text{Stress @ } b\text{–}b}{16{,}000} = \frac{15.5}{33} \qquad \text{Stress @ } b\text{–}b = 7520\#/\square''$$

* 4 vertical lines in the horizontal plate are not quite sufficient. The rivets might be spaced a little farther apart vertically and a few less used, as a possible arrangement.

The average stress for the horizontal plate is approximately

$$\frac{12{,}120 + 7520}{2} = 9820\#/\square''.$$

The total moment of resistance may then be expressed as
M_r = Vertical Plate + 2 Horizontal Plates

$$= \frac{s \cdot t \cdot d^2}{6} + 2 \times \text{ave. stress} \times \text{area} \times \text{arm}$$

$$= \frac{s \cdot t \cdot d^2}{6} + 2 \times s \text{ (ave.)} \times 9 \times t \times 20.5.$$

$$M_r = \frac{7520\, t \times (31)^2}{6} + 2 \times 9820 \times 9\, t \times 20.5 = 4{,}900{,}000''\#$$

t (for both plates) = 1.01" Use splice Pls. ½" thick.

The sizes are indicated in Fig. 134. These are determined by maintaining 1½" edge distance for the rivets.

Flange Angle Splice. Since angles are usually available in 60'-0" lengths and sometimes longer, no splicing of the flange angles is required.

Cover Plate Splice. Assuming that the maximum length of cover plate available is 36'-0", the inside cover plate would have to be spliced. The lengths of the cover plates are 46'-0", 36'-0" and 26'-0". The second cover plate, therefore, projects by the splice line 5'-0" approximately and it may be used as a splice plate. This method is always desirable, as it avoids an awkward splice plate on the top of the girder.

A_{CN} (the net area of the plate to be spliced) = 12.0□"

$$12.0 \times 16{,}000 = 192{,}000\#$$

Single Shear ⅞" φ rivet = 7220#

$$\frac{192{,}000}{7220} = 26.6.$$

Use 28 Rivets.

There is no intervening plate between the plate spliced and the splicing plate so that no extra rivets will have to be added to those theoretically required (Art. 74).

$$\frac{28}{2} = 14 \text{ rivets each side of the center line of girder.}$$

13 spaces @ 3" = 39". Hence these rivets may be easily placed in the 5'-0" available.

Erection Seat. An erection seat will be required to facilitate the erection. The weight of the girder and the time of the erection may be calculated as follows:

2-18 × ⅝ C.Pl. × 46'-0"	= 1.5 × 46 × 30.6 × 2 =	4,240#
2-18 × ⅝ C.Pl. × 36'-0"	= 1.5 × 36 × 30.6 × 2 =	3,310
2-18 × ½ C.Pl. × 26'-0"	= 1.5 × 26 × 25.5 × 2 =	1,990
4-8 × 8 × ¾ Ls × 59'-0"	= 4 × 45 × 59.0 =	10,620
1-66 × 5/16 Web Pl. × 59'-0"	= 5.5 × 59 × 22.95 =	7,450
		27,610
Add 10% for the weight of details such as stiffeners, etc.	=	2,760
		30,370

The reaction due to this weight $= \frac{30{,}370}{2} = 15{,}180\#$. The standard erection seat is a 6 × 4. The controlling value of the rivets as in Fig. 136 is 7220#.

$$\text{No. required} = \frac{15{,}180}{7220} = 2.1.$$

Use 4 for practical reasons.

Assuming uniform distribution of the reaction and a 16″ width of bearing, the load on a 1″ strip is $\frac{15{,}870}{16} = 1000\#/\text{in.}$, approximately. The load is assumed to be concentrated at the center of the bearing length. The moment arm, referring to Fig. 136 (c), $= 2\frac{1}{4} - \frac{5}{8} - \frac{1}{2} = 1\frac{1}{8}''$.

$$M = 1000 \times 1\tfrac{1}{8} = 1125''\#\text{ for a 1'' strip.}$$

$$1125 = \frac{s \cdot b \cdot t^2}{6} = \frac{16{,}000 \times 1 \times t^2}{6} \qquad t = 0.66''$$

Use 6 × 4 × $\frac{11}{16}$ Seat Angle 1′-4″ long

The seat angle is riveted $\frac{1}{4}''$ low to the column face so that when the end connection of the girder is riveted in place, the girder will be free from the erection seat. The latter may or may not be left in place depending upon whether it interferes with the finish.

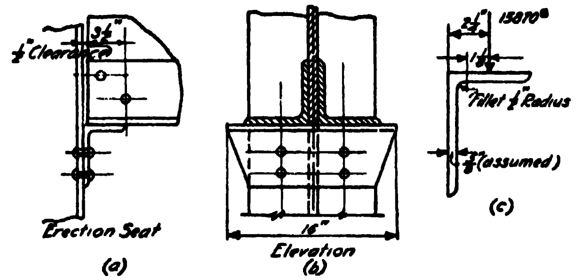


FIG. 136

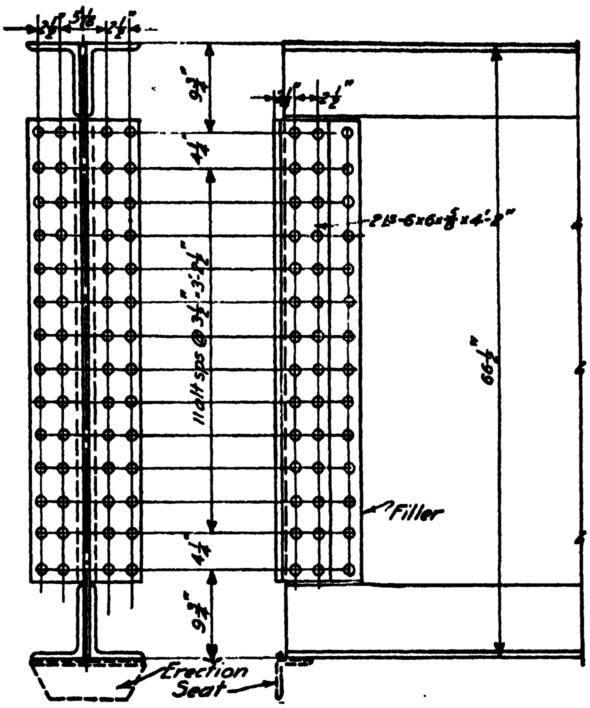


FIG. 137

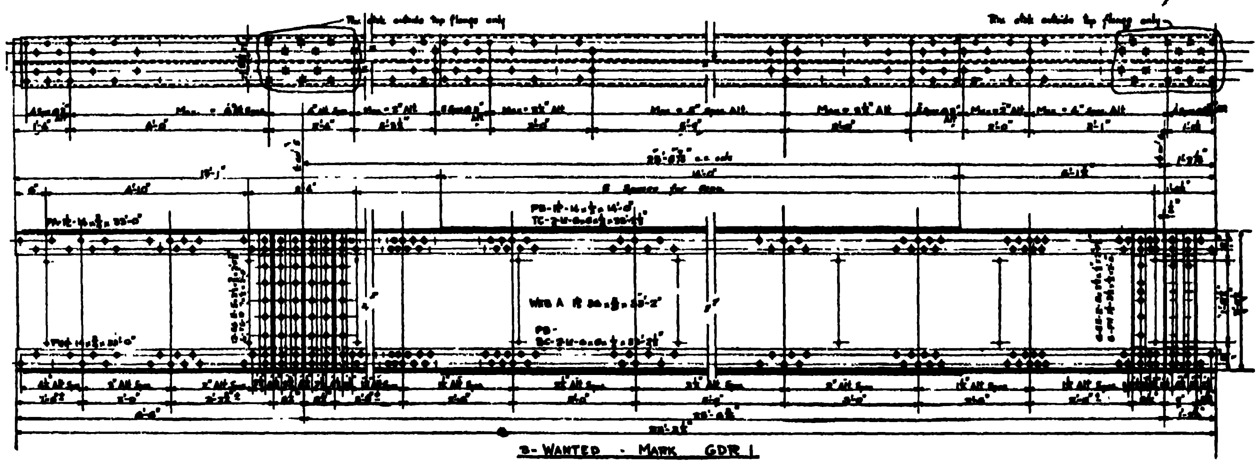


FIG. 138*

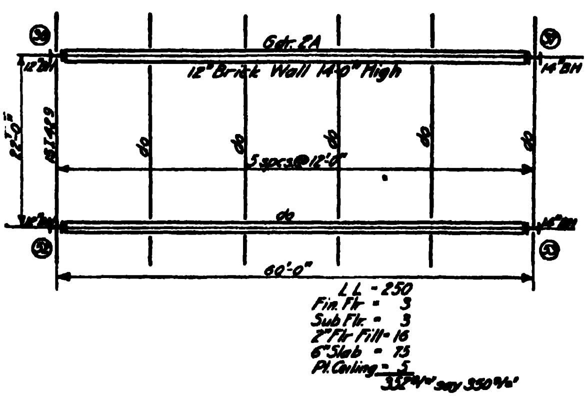


FIG. 139

End Connection. The end reaction = 324,000#. For the web connection, the controlling value of the rivet is 14,430#.

$$\frac{324{,}000}{14{,}430} = 22.5 \text{ or } 23 \text{ rivets are required.}$$

For the rivets in the outstanding legs of the connection angles, the controlling value for field rivets is 6020#.

$$\frac{324{,}000}{6020} = 54 \text{ required.}$$

Figure 137 shows the end connection. 27 rivets are used in each leg of each angle for symmetry. Figure 138 shows a typical shop drawing of a similar girder, but not the one designed.

Prob. 76a. Design a plate girder for the conditions of loading shown in Fig. 139.

* Courtesy of the Eastern Bridge and Structural Company, Worcester, Mass.

Section 5J

PLATE GIRDER DETAILS

77. General.

"No engineer is thoroughly competent to design steel structures unless he has had previous experience in detailing. For this reason fabricating companies seldom employ as designers men who have not spent three or more years in making working drawings. Instructions for a drafting room can usually be read with profit by a designer."* This is particularly true in the case of plate girders. An inexperienced designer may call for details which are entirely wrong, or perhaps impossible, from a fabricator's viewpoint. Some of the important details are discussed in this section for this reason. In addition, many points are mentioned in the previous discussion which border on the nature of details, purposely placed there.

78. Beam Connections.

If a **stiffener** is to be **used** to serve jointly as a beam connection, the flanges of the beam may be blocked off on one side, and the web of the beam may be placed against the outstanding leg of the stiffener angle. A field connection may be made as shown in Fig. 140 (*a*). The stiffener is located with respect to the length of the girder by "witnessing"† its gauge line. The dimension must be such that the center line of the beam is in the correct position. It is often advisable to space at least one rivet through the web of the girder unsymmetrical with the depth to prevent the possibility of placing the stiffener upside down. In Fig. 140 (*a*), only the bottom end of the stiffener need be milled. Some fabricators mark the bearing end, B.E., to make sure that the stiffener is properly placed. The above method is satisfactory for light beams but when heavy beams frame in, some other form of connection is used, because it is difficult to support the beam and drive the rivets at the same time.

Illustrative Prob. 78a. If the beam in Fig. 140 (*a*) is a 12 I 31.8 having an end reaction of 14,000#, and the stiffener is a 4 × 3 × ⅜ angle, how many ¾" field rivets are required?
Web thickness of 12 I 31.8 = ⅜".
Bearing of ¾" rivet on ⅜" metal (either the beam web or the angle in this case) = 5630#. Single shear = 4420#.

$$\frac{14{,}000}{4420} = 3.2 \qquad \text{Use 4 field rivets as shown spaced 3'' o.c.}$$

Standard connection angles on beams may be connected to plate girder webs, as illustrated in Fig. 140 (*b*), if the clearance below the flange allows the riveting. The same objection, however, is

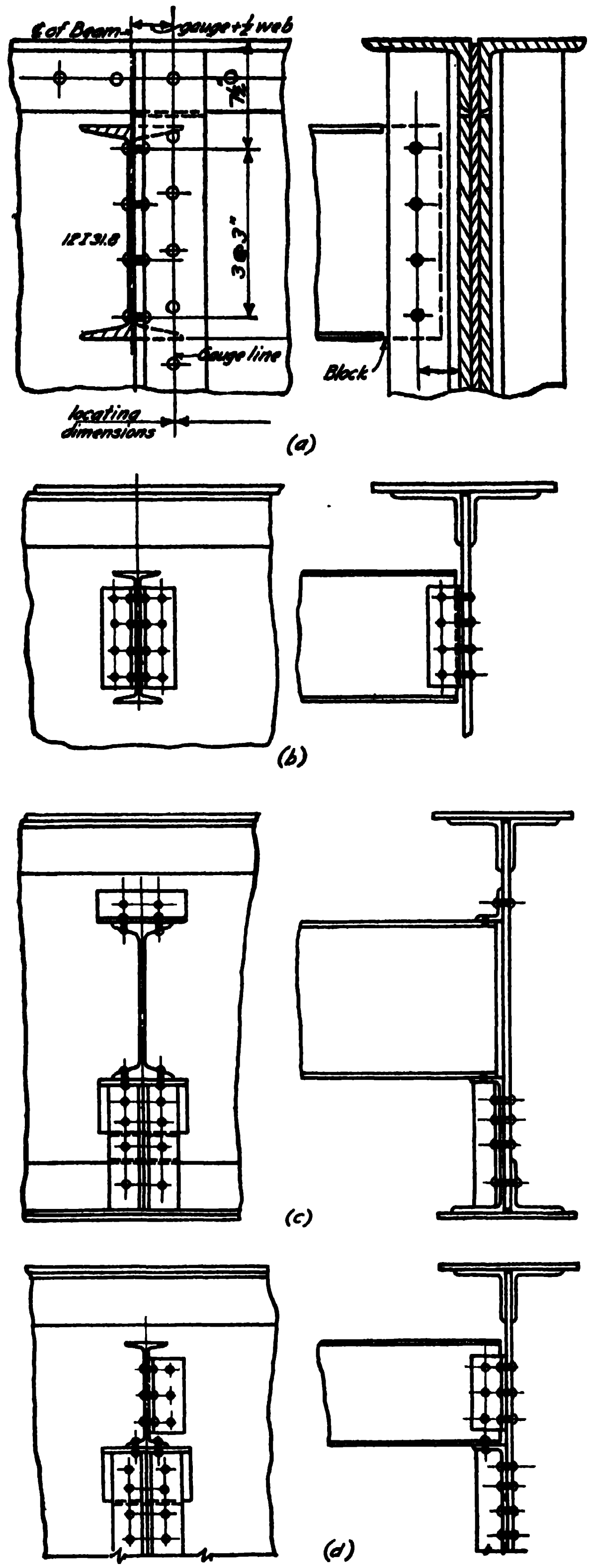


FIG. 140

* From an article by R. Fleming, Engineering News Record, June 3, 1920.
† The lines which tie the dimensions of details to each other and which bound other dimensions are called "witness lines."

present here, as for beams framed into the stiffeners, namely, that difficult erection is encountered in many cases.

Figure 140 (*c*) shows a common framing detail for a beam, which is the standard **seat angle** with the stiffeners under it. The design is similar to that for seat angles for beams on columns (see Index).

Illustrative Prob. 78b. If the beam in Fig. 140 (*c*) is an 18 I 54.9 having an end reaction of 50,000#, how many rivets are required? ¾" field rivets.

The size of the stiffener angles is determined in the usual way (Art. 55).

Single shear = 4420#.

Bearing on ⅜" metal = 5630#.

$\frac{50{,}000}{4420} = 11.3$ Use 12 rivets.

4 are placed in seat angle, as shown.

8 more must be placed below seat angle.

The spacing must not exceed 6".

Use 6 × 4 × ½ seat angle × 0'-6"
2-4 × 3 × ⅜ stiffener L^s.

Use fillers to take up the spaces between the thickness of the flange angle and the seat angle and the web plate. In such a case, one filler may be eliminated by making the thickness of the seat angle the same as that of the flange angle. Use 3½ × 3 × ⅜ top clip (T.C.) to secure beam laterally.

A side clip may be used instead of a top clip in certain instances, such as when clearance controls, as illustrated in Fig. 140 (*d*).

Prob. 78c. If the beam in Fig. 140 (*a*) is an 18 I 54.7 having an end reaction of 22,000#, and the stiffener is a 5 × 3½ × ⅜ angle, how many ¾" field rivets are required?

Prob. 78d. How many ¾" shop rivets are required in Illustrative Prob. 78*b*?

79. Stiffener Details.

The following **details**, controlled by stiffeners, are important:

(1) The outstanding legs of exposed stiffeners in double pairs should be stitch-riveted 12" o.c., or else they should be spaced at least 2" apart to allow for field painting and inspection.

(2) Stiffeners should be ordered ¼" longer than actually required, in order to allow for milling.

(3) Crimped stiffeners should be ordered to a length equal to the finished length plus the thickness of each angle over which it is offset.

Stiffeners control certain **rivet spacings**, as illustrated in Fig. 141. The spacing of the flange rivets in (*a*), either side of the stiffener, is controlled by the dimensions of the rivet die (Art. 25). The clearance for rivets in crimped angles must be maintained, as in (*b*). The cover plate rivets must not foul the stiffeners and clearance must be provided so that the rivets can be driven, as shown in (*c*).

When stiffeners are placed **under columns**, their outstanding legs should be directly in line with the flanges of the columns, as shown in Fig. 142. If the web of a column happened to be at right angles to the web of a girder, it may be better to use two plate girders with the webs under the flanges of the column or to use a box girder (Art. 84).

Intermediate stiffener angles are generally faced so that their outstanding legs are toward the center line of the girder, although this is a matter of local practice and it makes no particular difference.

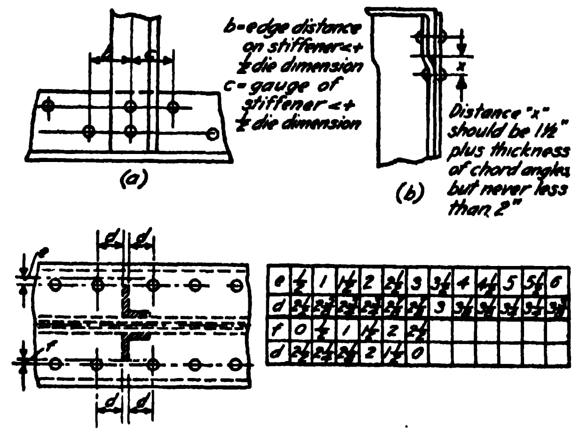


FIG. 141

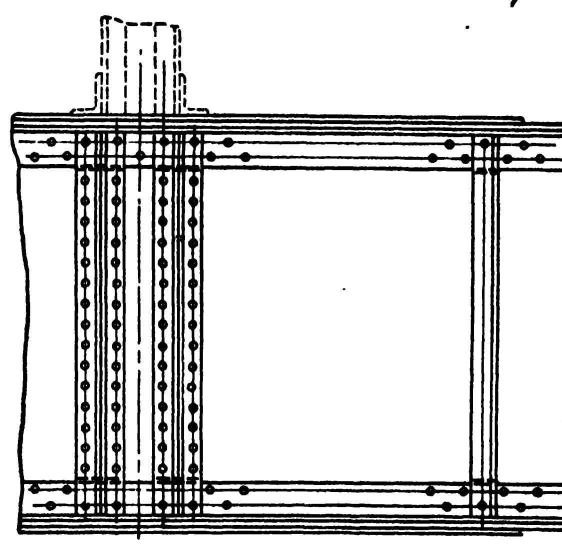


FIG. 142

80. Filler Details.

Fillers are used when a stress is transferred from one plate to another which is not in direct contact with the first. They are classified as loose or tight according to the method of riveting them. In Fig. 143 (*a*), the **filler** is **loose** because it is free to move with any deformation of the joint. It should never be used where a transfer in the direction of stress occurs, as dangerous secondary stresses are developed. Figure 143 (*b*) shows one type of a **tight filler**, that is, the filler is connected by in-

dependent rivets and it is not free to move. In the latter type, in addition to the rivets required for direct stress, rivets are required for the secondary stress.* There are various rules which have been used to approximate the extra rivets required, and a common one is to add 50% to the number of rivets required for the direct stress, for each intervening filler. Such a rule takes no account of the relative thicknesses of the plates and the filler, in offering bearing resistance. It is also assumed that the rivets distribute the stress from plate *a* in Fig. 143

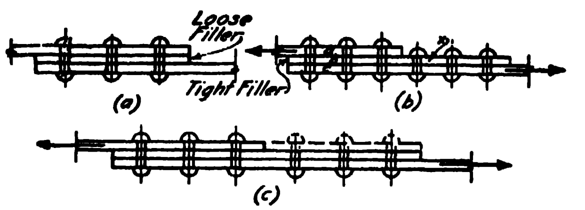


FIG. 143

(*b*), uniformly over plates *b* and *c*, as if they were one piece. A more exact solution would be to distribute the rivets in proportion to the thicknesses of the plates, especially if the thickness of *a* is less than the thickness of *b* or *c*. In any case the load should be assumed to travel in the direction which gives the greater number of filler rivets. The top plate may be extended, as indicated by the dotted lines in Fig. 143 (*c*), if desired. This virtually makes a loose filler but with a sufficient number of rivets to develop the stresses.

The following suggestions are useful in determining filler **dimensions**:

(1) Loose fillers should be made the same width as the leg of the stiffener adjacent to the web plate.

(2) A clearance of $\frac{1}{4}''$ is generally maintained between the ends of a filler bar and the toes of the flange angles, top and bottom, except for exposed work. In the latter, the length is made the same as the clear distance between the toes of the top and bottom flange angles, in order to avoid rain pockets. Care should be taken to provide against the overrun of the flange angles.

(3) Fillers less than $\frac{3}{16}''$ thick are not generally used.

(4) When girders are laterally unsupported, the top and bottom flange angles may be of different thicknesses. This necessitates judgment relative to the thickness of the filler. The following is suggested:

Difference	*Filler*
$\frac{1}{16}''$	use filler of either angle thickness.
$\frac{1}{8}''$	use mean thickness.
$>\frac{1}{8}''$	use two fillers of different thicknesses.

(5) In case of $\frac{1}{32}''$ differences in the thicknesses of metal, use a filler to the nearest $\frac{1}{16}''$ above, as the paint and scale on the metal will tend to make up the difference.

81. Framing Details at Columns.

When an end of a plate girder is to be supported by a column, it may be **framed into the face of the column** or it may run over the top. In the first case, the common method is to use an erection seat and a pair of special connection angles (Art. 29), as shown in Fig. 144 (*a*). A typical set of design computations is given in the last part of Art. 76 and the details resulting are shown in Fig. 137. An alternate detail is shown in Fig. 144 (*b*), in which the end reaction is carried by a seat angle with stiffeners under it. This method involves more material, as stiffeners are required for

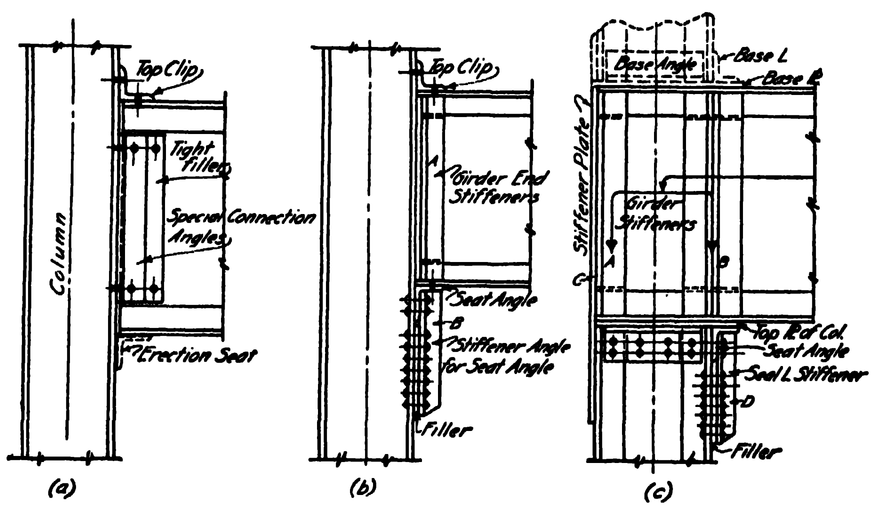


FIG. 144

the girder and the seat also. The seat angle and its stiffeners are shop riveted to the column so that they become a part of that member (for typical design, see Index). These stiffeners often affect the details with respect to the fire protection materials. In any case, a combination of the special connection angles, as in Fig. 144 (*a*), and a seat angle detail, as in Fig. 144 (*b*), should never be used

* The method of design which is nearest to correct is that for designing pin plates (see Index), although such a method is considered too conservative by some designers for this detail.

if it can be avoided (Art. 29). The stiffener angles A and B in (b) have to be very heavy, as only one pair can be used to develop the end reaction.

When the end of the girder runs **over the top of a column,** a detail similar to that in Fig. 144 (c) may be used. This is considered by many engineers to be a good detail to use even if the column continues above the girder, as indicated by the dotted lines. A stiffening plate, C, should be used to make the connection more rigid and to aid in reducing the eccentricity in the column as much as possible. When the girder deflects, as it will, more load is thrown into the stiffeners B than into those at A. The exact amounts of the load are indeterminate, but a conservative rule is to design the stiffeners B and D for two-thirds the end reaction and the stiffeners A for one-half the end reaction. Those at A and B may be designed as described in Art. 56, while those at D may be designed as described for seat angles (see Index).

82. Ends of Girders Bearing on Walls.

When an end of a plate girder rests upon a wall, the **arrangement of** the end **stiffeners** is important. They may be placed as shown in Fig. 145 (a) or (b). In (a), the stiffeners A and B are placed so that their outstanding legs are flush with the outer and inner edges of the bearing plate, respectively. The web

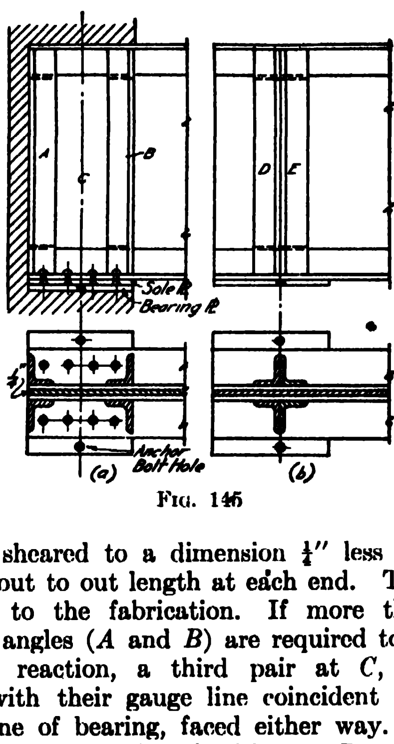


FIG. 145

plate is sheared to a dimension ¼″ less than the finished out to out length at each end. This gives a finish to the fabrication. If more than four stiffener angles (A and B) are required to develop the end reaction, a third pair at C, may be placed with their gauge line coincident with the center line of bearing, faced either way. In Fig. 145 (b), the two pairs of stiffeners, D and E, are placed symmetrically about the center line of bearing. Many engineers consider this to be a better detail, as the loads on the stiffeners in (a) are more or less indeterminate. In either detail, the design is accomplished by the use of the principles outlined in Art. 56. For detail (a), a conservative rule is to design stiffeners B for two-thirds the end reaction and stiffeners A for one-half the end reaction.

When the end of the girder is detailed to sustain the end reaction, it still remains to distribute the load properly on the wall. **For light girders,** it may be possible to use a **simple bearing plate** similar to those for rolled beams (Art. 15). In the usual case, however, the thickness of plate required is excessive, and a **sole plate** is riveted to the bottom of the girder, as shown in Fig. 145, to provide a portion of the required thickness. Such a plate also ties the outstanding legs of the bottom flange together at the bearing, and in addition, it provides a place for the anchor bolts. A typical design is given near the end of Art. 39.

Plate girders which are connected to columns are given rigidity by the end connections (Art. 29). When a girder rests upon a wall, **anchor bolts** are used in many cases to aid in making the end secure against motion. They are used particularly when an uplift is exerted at the end of a girder by unbalanced loads caused by cantilevers. The size is arbitrarily made 1″ when only a question of lateral rigidity is concerned, but when uplift is possible, they should have a sufficient net section to offer a safe tensile resistance to such action. The bolts in the latter case must have a length of embedment which will engage a sufficient weight of masonry to offset the uplift.* At the lower ends of the bolts, single washers or a combined plate washer must be used which will have area enough to develop the tensile strength of the bolts by bearing on the masonry. The thickness of the washers must be great enough to safely resist the bending induced in them by the bearing action (Art. 13, Book 1).

The **holes** for anchor bolts are generally drilled $\frac{5}{16}$″ **greater than** the **diameter of** the **bolts** to allow for the placing of the girder after the bolts have been set.

When a girder is exposed to the weather, an appreciable change in its length occurs, due to expansion and contraction, if the span exceeds 50′-0″. The change in length may be computed by using the coefficient of expansion. For girders which are confined to the interiors of buildings, the change of length is naturally much less, and no provision is made in many cases for short spans, and particularly when the girder is encased with fire protecting materials. For other cases of interior work in which the girders are exposed, the

* This does not mean below the bearing of the girder, except for roof girders, as the upper story masonry assists in providing weight in the usual case.

change may be calculated upon a basis of a smaller range of temperature. Provision for the longitudinal movement of a girder caused by such action may be made by the use of **slotted holes,** as shown in Fig. 146 (*a*). Incidentally, these give extra allowance for any inaccurate setting of the bolts. The length of the slot is dependent upon the change of temperature, and it should allow movement in either direction from the mean position of the bolt.

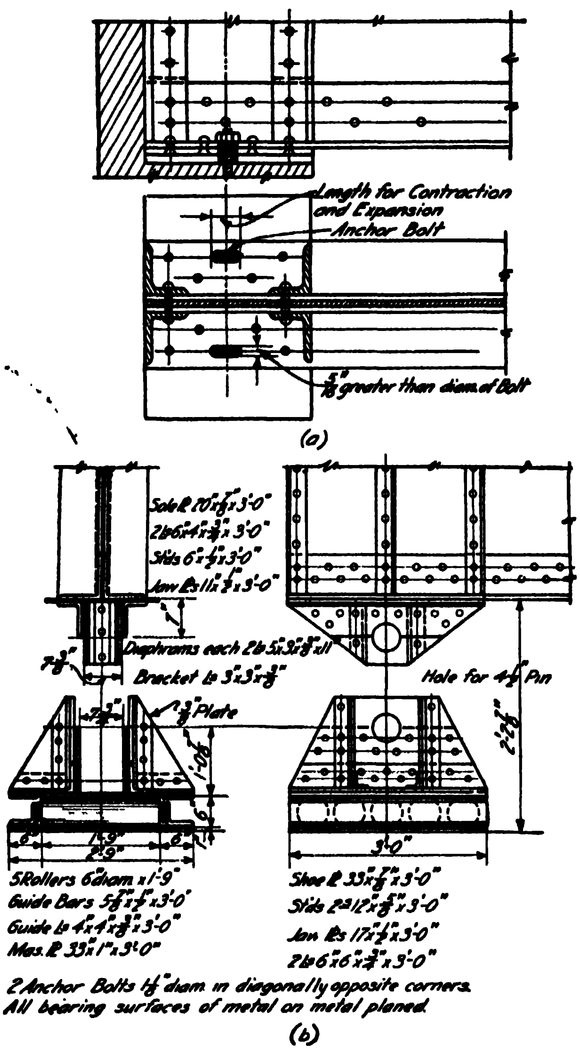


FIG. 146

A rule of thumb employed instead of using calculations is to allow a **length of ⅛" for every 10'-0" of span.** For very heavy girders, **rollers** may be used, as shown in Fig. 146 (*b*), but these are not common for girders in building work.

Illustrative Prob. 82a. If the anchor bolts are 2-1¼"φ and the girder span is 70'-0" and is exposed to outside temperatures what size of slotted holes should be used?

Size of anchor bolts = 1¼". Drill holes $1\frac{9}{16}$".
Low temperature say −20°
High temperature say 100°
range = 120° = T

k for steel = 0.0000065/° F.
$e = l \cdot T \cdot k$
$e = 70 \times 12 \times 120 \times 0.0000065 = 0.66''$ say $\frac{11}{16}''$
$1\frac{9}{16} + \frac{11}{16} = 2\frac{1}{4}''$. Make slotted holes 2¼" long.
If the rule of thumb were used, $e = 7 \times \frac{1}{8} = \frac{7}{8}''$.

For very **large reactions and limited wall bearings,** a sole plate and wall plate combination may be insufficient, as the thickness of the sole plate is limited by its riveting (Art. 23), and the thickness of the wall plate should ordinarily conform to stock sizes, say, 1½" thick as a limit. A **rolled steel slab** may be employed instead of a bearing plate in such an instance (see Index), but its use may mean delay, as these slabs are usually mill shipments. An alternate method is to use a **grillage.**

Illustrative Prob. 82b. Design an arrangement for a wall bearing of a plate girder with a 60 × ½ web plate and 8 × 8 × ⅞ flange angles. 24" brick wall, 1 : 3 P. C. mortar, ⅞" rivets. End reaction = 200,000#.

$$\text{Allowable bearing} = 250\#/\square''$$

$$\text{Required area} = \frac{200{,}000}{250} = 800\square''$$

Maximum available bearing length = 24 − 4 = 20".

$$\text{Bearing width} = \frac{800}{20} = 40''.$$

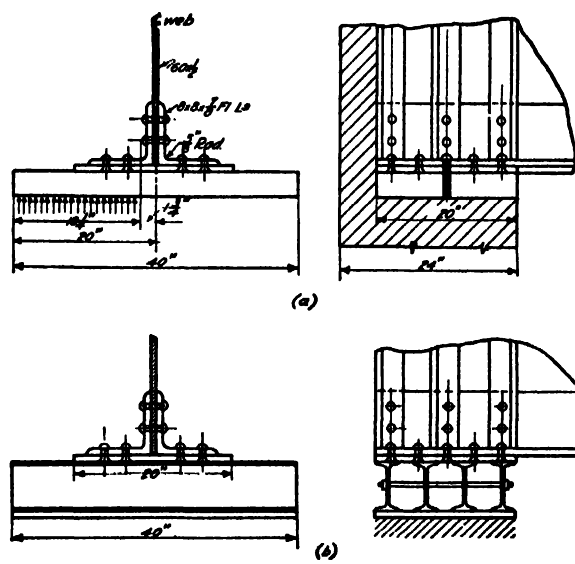


FIG. 147

Referring to Fig. 147 (*a*), the projection beyond the toe of the fillet of the flange angle is $20 - (\frac{1}{4} + \frac{7}{8} + \frac{5}{8}) = 18.25''$.

$$M_e \text{ for a 1'' strip} = \frac{250 \times (18.25)^2}{2} = 41{,}600''\#$$

Use 20 × ⅞" sole plate × 1'-8".

M_r of flange angle leg and sole plate combined $= \dfrac{s \cdot (1) \cdot t^2}{6}$

$$= \frac{20{,}000 \times 1 \times (\frac{7}{8} + \frac{7}{8})^2}{6} = 10{,}200''\#.$$

Moment still to be carried $= 41{,}600 - 10{,}200 = 31{,}400''\#.$

$$t = \sqrt{\frac{31{,}400 \times 6}{20{,}000}} = 3.08''$$

Use $3\frac{1}{4}''$ Rolled Steel Slab.
20″ wide, 3′-4″ long.
$20 \times \frac{7}{8}$ sole Pl. $\times$ 1′-8″.

Bearing under sole plate $= \dfrac{200{,}000}{20 \times 20} = 500\#/\square''$ O.K.

Alternate design.

Total moment about toe of fillet, Fig. 147 (*b*), is
$31{,}400 \times 20 = 628{,}000''\#$

$$\frac{I}{c} \text{ required} = \frac{628{,}000}{16{,}000} = 39.2''^3$$

20″ width available. Try 4 small beams.

$\dfrac{39.2}{4} = 9.8''^3$ per beam. Try 7 I 15.3.

flange width $b = 3.66''$. $4 \times 3.66 = 14.64''$
$20.00 - 14.64 = 5.36''$. 3 spaces between beams

$\dfrac{5.36}{3} = 1.78$ O.K. for concreting. Space beams $5\frac{3}{8}''$ o.c.

Maximum shear in grillage beams

$$v = \frac{W}{L} \times \frac{L - a}{2} \times \frac{1}{n \cdot d \cdot t}$$

$L = 3.33'$, $a = 1.67'$, $n = 4$, $d = 10''$, $t = 0.25''$

$$v = \frac{200{,}000}{3.33} \times \frac{3.33 - 1.67}{2} \times \frac{1}{4 \times 10 \times 0.25} = 5000\#/\square'' \text{ O.K.}$$

allowable $= 10{,}000\#/\square''$.

Buckling* $f_b = 16{,}000 - 120\dfrac{d}{t} = 16{,}000 - \dfrac{120 \times 10}{0.25}$

$f_b = 11{,}200\#/\square''$ allowable

$$f_b = \frac{W}{n\left(12\,a + \frac{d}{2}\right)t} = \frac{200{,}000}{4\left(20 + \frac{10}{2}\right)0.25} = 8000\#/\square'' \text{ O.K.}$$

Use $20 \times \frac{7}{8}$ sole plate $\times$ 1′-8″
Use 4-7 I 15.3 $\times$ 3′-4″

Use $20 \times \frac{3}{4} \times$ 3′-4″ distributing plate under beams.
Use 1″ gas pipe and $\frac{3}{4}''$ rod separators (Art. 32).

Prob. 82c. Determine the size of slotted holes for two 1″ϕ anchor bolts for a girder span of 80′-0″ exposed to outside temperatures.

Prob. 82d. Determine an arrangement for the end bearing of a plate girder on a 20″ brick wall, 1 : 3 P. C. mortar, if the end reaction is 70,000#. Plate girder section $48 \times \frac{3}{8}$ web plate, flange angles $5 \times 3\frac{1}{2} \times \frac{5}{8}$. $\frac{3}{4}''$ rivets. Use sole plate and bearing plate if possible.

Prob. 82e. Design an arrangement for a wall bearing of a plate girder with a $60 \times \frac{1}{2}$ web plate and $6 \times 6 \times \frac{3}{4}$ flange angles. 24″ brick wall, 1 : 3 P. C. mortar. $\frac{7}{8}''$ rivets. End reaction = 150,000#.

83. Camber.

When a structural member has a long span, an optical illusion occurs and the member appears to sag. Quite frequently a plate girder is bowed upward slightly in the fabrication to offset this appearance. The amount of such distortion is called camber. It is also advantageous in that a girder will not deflect unduly below a horizontal when it is fully loaded, and consequently protection against cracking is afforded any materials fastened to the girder. In the fabrication, the girder is blocked up at the center the required amount when it is being assembled. The amount of camber may be equal to the maximum deflection as calculated (Art. 51), or it may be made only a portion of that value, depending upon judgment. The following rules are sometimes employed to determine the amount of camber for girders of spans greater than 50′-0″:

1/1000 to 1/1500 of the span, or
1/16″ per 10′-0″ of the span.

When a girder is cambered, the rivet spacing must be adjusted. The web plate is usually kept straight and the variation of the pitch of the rivets in the vertical legs of the flange angles is determined by the template maker. The top flange angles are detailed slightly longer than those at the bottom, the difference being determined by the difference in the lengths of the two arcs, as illustrated in an exaggerated way in Fig. 148 (*a*). The rivets in web splices are placed slightly farther apart at the top than at the bottom in order to aid the fabrication, as in (*b*).

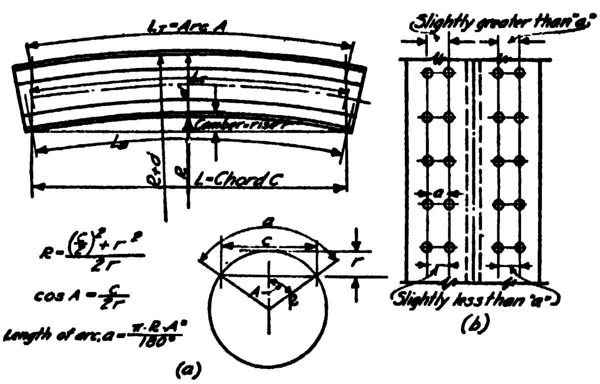


FIG. 148

Some engineers believe that cambering is practically useless and an unnecessary expense. The authors believe that cambering should be used in certain cases such as when the soffit of the girder is exposed. Care must be used on the job to see that any fire protecting materials and plaster attached to the soffit in such a case follow the established curve of the bottom of the girder, for otherwise the principal object of the camber is destroyed. When a suspended ceiling is used, cambering of a plate girder is not necessary if the deflection is within safe limits.

Prob. 83a. What camber should be allowed for a girder of 70′-0″ span?

* This is one instance where buckling should be carefully investigated.

SECTION 5K

MISCELLANEOUS GIRDERS

84. Box Girders.

A box girder is one in which more than one, and usually two, web plates are used, as illustrated in Fig. 149. Some of the instances in which they are used are as follows:

(1) When the available depth for a girder is limited, such as over assembly halls, lobbies and so on,

(2) when the shear is large,

(3) in foundations where the exterior columns are carried by cantilever construction,

(4) to distribute loads from heavy columns or to support thick masonry walls,*

(5) for the bridge girders of electric traveling cranes,

(6) when a girder lacks lateral support, the broad flange of a box girder offers stiffness, as it has a well distributed horizontal area of steel.

A box girder is uneconomical as compared with a single plate girder. There is a gain of only about 12% over the single plate girder when a box girder with the same flange steel and depth is used. Such a member is clumsy and expensive to fabricate and it is not accessible for field painting and inspection. The flange rivets are limited in strength to single shear, which also increases the cost of the girder.

In certain cases, such as when crane girders have walkways on the sides, torsional moment is introduced unless provisions are made to eliminate it. It is better to avoid torsion than to provide for it. In the case of a walkway, a stiffening girder, attached by lattice bars (see Index), may be used to avoid any torsional moment. The bottom flange forms the support for the walkway and the web system serves as a railing. If a stiffening girder is not used, overturning moment is developed by the walkway. If the latter consists of two or three planks supported by light brackets, the moment is small and it may be neglected. If the walkway is of steel or if a motor is located upon it, a considerable moment is developed and the torsional stresses must be kept within safe limits. The theory of such stresses acting on a rectangular prism is very vague. The following is suggested by Mr. R. Fleming:

SPECIFICATION CLAUSE†

Torsional Moment "The torsional moment in box girders shall be assumed to produce vertical shears in the web plates and horizontal shears in the cover plates. These shears are assumed to produce no bending in the members in which they act and are to be regarded as pure shears. The torsional moment shall be divided equally between the web plates and the covers, that is, $S_1 \cdot b = S_2 \cdot h$ (Fig. 149); where S_1 and S_2 are the total shears in web plates and cover plates, respectively.

"Full-length diaphragms shall be placed at all points of applied eccentric loads to distribute the shears properly to the component parts of the girder. Sufficient diaphragms shall be used throughout to maintain the true form of the girder.

"The unit stress in the connection between the web plates and the covers, produced by the torsional moment alone, shall be taken as equal to $S_1/h = S_2/b$. This stress acts in a direction parallel to the axis of the girder."

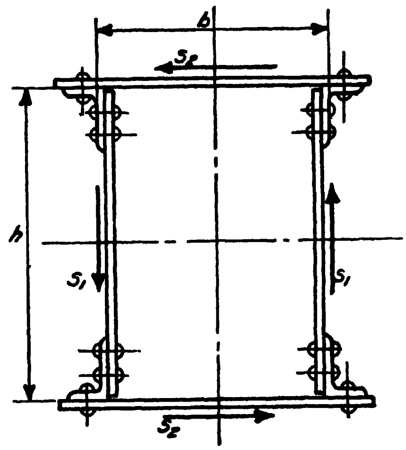


FIG. 149

At least one full-length cover plate should be used to hold the webs of box girders together and to aid in providing stiffness by distributing the stresses. The distance between the webs must be within a reasonable figure, so that the breadth of the cover plate will not be too great compared with its thickness. If more than one cover plate is used, the cut-off points of the outside plates may be determined in the usual way (Art. 53). Stiffeners should be used on both sides of the web plates and proportioned and spaced as described in Sect. 5E. Diaphragm separators should be used at any interior columns (Art. 33). Field connections for beams should be preferably made with through bolts. Short bolts may be used if hand holes are provided (Art. 33). The assembly of a box girder must be in a definite manner in order that the rivets may be conveniently driven. The two web plates with their respective flange angles and stiffeners are first riveted together as individual units and then they may be tied together by riveting the

* Two usual plate girders side by side are considered better by many engineers for the support of brick walls more than 12" thick. The flanges are very often braced, but this does not approach the expense involved for box girders.

† Developed by the Chicago engineering office of the American Bridge Co.

cover plates. Figure 150 shows a typical girder detailed.

Illustrative Prob. 84a. Design a box girder to carry the loading shown in Fig. 151 (*a*). The depth is limited to 30″ on account of headroom. A 20″ width of soffit is also desired as a maximum.

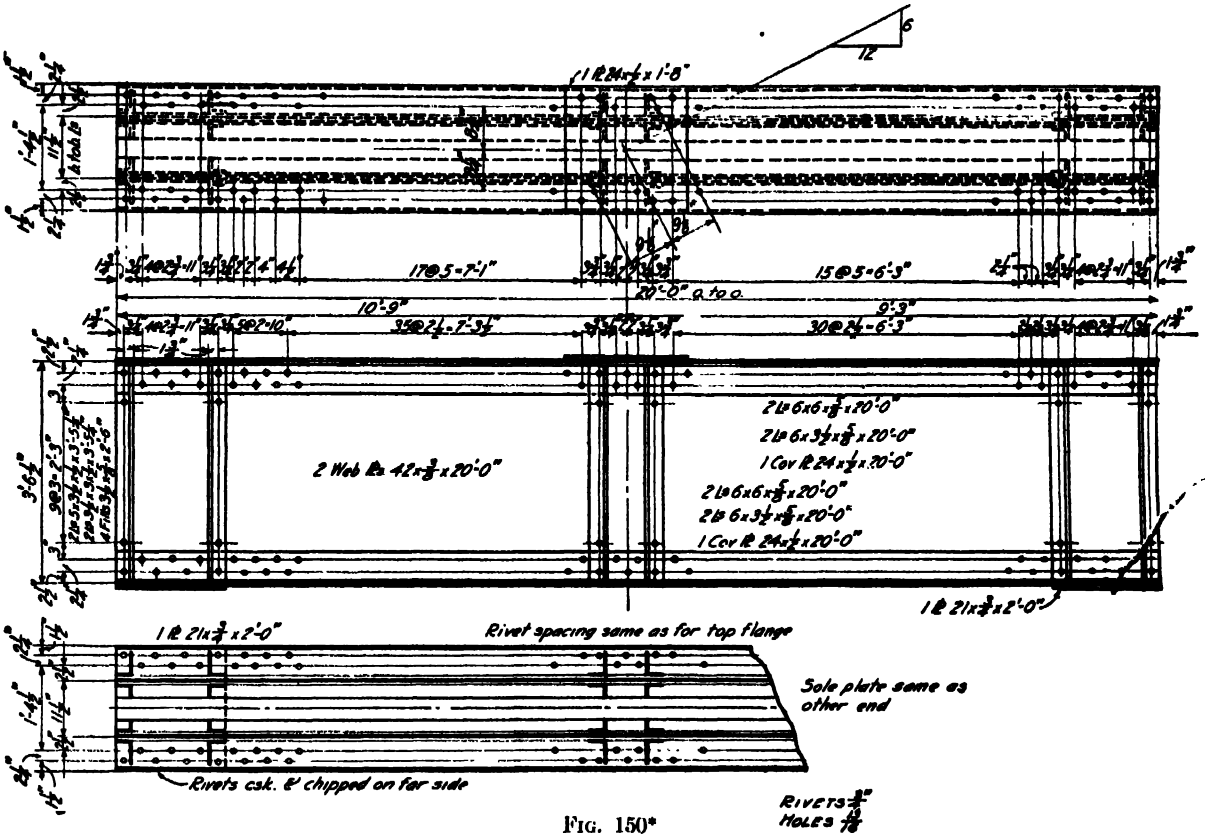


FIG. 150*

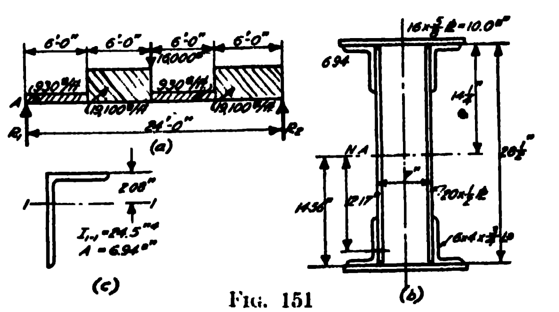


FIG. 151

$930 \times 6 \times 3 + 19{,}100 \times 6 \times 9 + 16{,}000 \times 12 + 930 \times 6 \times 15 + 19{,}100 \times 6 \times 21 - 24\,R_2 = 0 = \Sigma M_A$

Σ loads = 255,760 $\qquad R_2$ = 155,300#

R_2 = 155,300 $\qquad R_1$ = 100,460#

R_1 = 100,460

930×6 = 5,580

94,880

$$\frac{94{,}880}{19{,}100} = 4.97$$

$6 + 4.97 = 10.97'$ from R_1
Point of 0 shear

$$M_{max} = 100{,}460 \times 10.97 - 930 \times 6 \times 7.97 - 19{,}100 \times \frac{(4.97)^2}{2} = 822{,}500'\#$$

$$A_{WN}\ (2\ \text{plates}) = \frac{155{,}300}{12{,}000} = 12.94\square''$$

$$A_{WN}\ (1\ \text{plate}) = \frac{12.94}{2} = 6.47\square''$$

$$A_W\ (1\ \text{plate}) = \tfrac{4}{3} \times 6.47 = 8.63\square''$$

Try 28″ depth (on a/c 30″ headroom)

$$\frac{8.63}{28} = 0.308''$$

$\frac{5}{16}''$ or $\frac{3}{8}''$ web plates could be used with intermediate stiffeners.

$$\frac{28}{60} = 0.466''.$$

If $\frac{1}{2}''$ web plates are used no intermediate stiffeners are required (see Specification page 86).

Use 2-28 × ½ Web Plates.

$M = 822{,}500 \times 12 = 9{,}870{,}000''\#$ Assume $d_e = 28''$

$$F = \frac{M}{d_e} = \frac{9{,}870{,}000}{28} = 352{,}000\#$$

$$A_{FN} = \frac{F}{f_s} = \frac{352{,}000}{16{,}000} = 22.0\square''$$

$$\tfrac{1}{8} A_W = \tfrac{1}{8} \times 28 \times \tfrac{1}{2} \times 2 = \frac{3.5}{18.5}$$

* Adapted from Bishop's "Structural Drafting"—John Wiley & Sons, Inc.

Test rivet pitch. Try $\frac{3}{4}''$ rivets.

$$p = \frac{R \cdot d_e}{V} = \frac{5300 \times 28}{155,300} = 0.96'' \text{ (too close).}$$

Use $\frac{7}{8}''$ rivets, $p = \dfrac{7220 \times 28}{155,300} = 1.3''$ O.K. if rivets are staggered.

This pitch is only required for a short distance at the right-hand end and is calculated at this time as a guide so that net sections may be calculated. About one-half of flange area should be in the flange angles. $\dfrac{18.5}{2} = 9.25\square''$

$$\begin{array}{ll} \text{Try } 6 \times 4 \times \frac{3}{4} = 6.94 & 2 \times 5.44 = 10.88\square'' \text{ O.K.} \\ \text{2 holes out} & \\ 2 \times 1 \times \frac{3}{4} = \underline{1.50} & \text{Use 2-6} \times 4 \times \frac{3}{4} \text{ flange } \llcorner^s \\ \qquad\quad 5.44 & \end{array}$$

$18.50 - 10.88 = 7.62\square''$ to be supplied by the cover plate. Try 16″ plate (on a/c 20″ soffit and 2″ F.P. each side).

$$\begin{array}{ll} 16 \times \frac{5}{8} \text{ Pl.} = 10.00\square'' & \\ \text{2 holes out} & \\ 2 \times 1 \times \frac{5}{8} = \underline{1.25} & \text{Use } 16 \times \frac{5}{8} \text{ cover plate.} \\ \qquad\quad 8.75\square'' & \text{Place web plates 7'' o.c. to} \\ & \quad \text{maintain 16'' width.} \end{array}$$

$$\frac{l}{b} = \frac{24 \times 12}{16} = 18 < 20 \text{ O.K.}$$

$$7 + \tfrac{1}{2} + 2 \times 2\tfrac{1}{2} = 12\tfrac{1}{2}'' \text{ o.c. between rivet lines}$$

$$\frac{12.5}{0.5} = 25 = \frac{l}{t} \text{ ratio for cover plate O.K.}$$

Taking moments about the neutral axis, to locate the center of gravity of the flange,

$$\frac{d_e}{2} = \frac{10 \times 14.56 + 2 \times 6.94 \times 12.17}{10 + 2 \times 6.94} = 13.2'' \quad d_e = 26.4''$$

$26.4'' < 28.5''$ O.K. for specification.

Check compression area.

$$F = 352,000 \quad f_c = 14,000\#/\square''$$

$$\frac{352,000}{14,000} = 25.1\square'' \text{ gross}$$

$$\begin{array}{rl} \frac{1}{8} A_W = & 3.50 \\ 2 \llcorner^s = & 13.88 \\ 16 \times \frac{5}{8} \text{ Pl.} = & \underline{10.00} \\ & 27.38\square'' \text{ O.K.} \end{array}$$

Check stress on extreme fibre. Calculate I

$$\begin{array}{ll} \frac{b \cdot d^3}{12} \text{ for 2 web plates} = \frac{2 \times 0.5 \times (28)^3}{12} & = 1828 \\ A \cdot d^2 \text{ for } 2 \llcorner^s \text{ net} = 5.44 \times 2 \times \overline{12.17}^2 & = 1610 \\ I \text{ for } 4 \llcorner^s = 4 \times 24.5 & = \quad 98 \\ A \cdot d^2 \text{ for } 2 \llcorner^s \text{ gross} = 6.94 \times 2 \times \overline{12.17}^2 & = 2058 \\ I \text{ for cover plates about own axes} & \\ \quad \text{neglected} & \\ A \cdot d^2 \text{ for plate net} = 1 \times 8.75 \times \overline{14.56}^2 & = 1853 \\ A \cdot d^2 \text{ for plate gross} = 1 \times 10.0 \times \overline{14.56}^2 & = \underline{2120} \\ \qquad\qquad I \text{ (total)} & = \overline{9567}''^4 \end{array}$$

$$M = \frac{s \cdot I}{c} = 9,870,000 = \frac{s \times 9567}{14.87} \qquad s = 15,300\#/\square'' \text{ O.K.}$$

The cover plate extends the full length of the girder. No intermediate stiffeners are required (see previous calculations). Stiffeners should be placed at the 16,000# concentrated load (see loading diagram) according to the details of the construction. (For typical design, see Art. 55.)

Flange Rivets.

$$p = \frac{R \cdot d_e}{V} = \frac{7220 \times 26.6}{155,300} = 1.3'' \text{ Use } 1\tfrac{1}{4}'' \text{ o.c., staggered.}$$

Spacing diagonally O.K. for $2\frac{1}{2}''$ gauge.

V @ 1′-0″ from $R_2 = 155,300 - 19,100 = 136,200$

$$p = \frac{7220 \times 26.6}{136,200} = 1.41'' \qquad \text{Use } 1\tfrac{1}{4}''$$

V @ 2′-0″ from $R_2 = 155,300 - 2 \times 19,100 = 117,100$

$$p = \frac{7220 \times 26.6}{117,100} = 1.64'' \qquad \text{Use } 1\tfrac{1}{2}'' \text{ pitch}$$

At left $\quad R_1 = 100,460\#$

$$p = \frac{7220 \times 26.6}{100,460} = 1.91'' \qquad \text{Use } 1\tfrac{3}{4}'' \text{ pitch}$$

Similarly, the pitch at other points could be calculated and an arrangement determined for the girder. A maximum of 6″ o.c. staggered should not be exceeded.

$$\text{For the cover plate rivets, } p = \frac{n \cdot R \cdot d_e}{V} \times \frac{A_{FN}}{A_{CN}}$$

$$p = \frac{2 \times 7220 \times 26.6}{155,300} \times \frac{23.13}{8.75} = 6.56''$$

Use 6″ o.c. staggered.

No splices are required in the web, cover plate, or flange angles, as the girder is only for a 24′-0″ span. If the girder rested upon walls, the design could be carried forward as illustrated in Art. 39, and if it framed into columns, the design would be similar to that described in Art. 76.

Illustrative Prob. 84b. If the box girder section in Illustrative Prob. 84a is subjected to a torsional moment of 350,000″#, what are the additional shears that should be provided for in the web plates and cover plates?

Torsional moment divided equally between the web plates and the cover plates

$$\frac{350,000}{2} = 175,000''\#. \qquad b = 9'' \qquad h = 28.5''$$

S_1 = vertical shear due to torsion in one web plate

$S_1 \cdot b = 175,000 = S_1 \times 9 \qquad S_1 = 19,450\#$

S_2 = horizontal shear due to torsion in one cover plate

$S_2 \cdot h = 175,000 = S_2 \times 28.5 \quad S_2 = 6160\#$.

The rivets must resist these forces in addition to those caused by the vertical loading.

Prob. 84c. Design a box girder to carry a load of 160,000# concentrated at the middle of a 32′-0″ span. Depth limited to 40″. Soffit limited to 20″.

85. Latticed Girders.*

Occasionally a girder is used in which the solid web plate is replaced by a system of diagonal web members. Angles are practically always used for the latter and such bracing may be either single or double, as illustrated in Fig. 152 (a) and (b). A common use of such a girder is to place it beside a long span crane runway girder (see Index) to act as a stiffening member and thus brace the crane girder against transverse thrusts caused by swinging loads and the like. The latticed girder is placed so that either its top or bottom flange is at the proper

* "Latticed girders" and "latticed trusses" are often used to mean the same thing. For a roof truss, the member will be called a triangular or Warren truss. A "latticed truss" will refer to a wood truss, only (see Book 1).

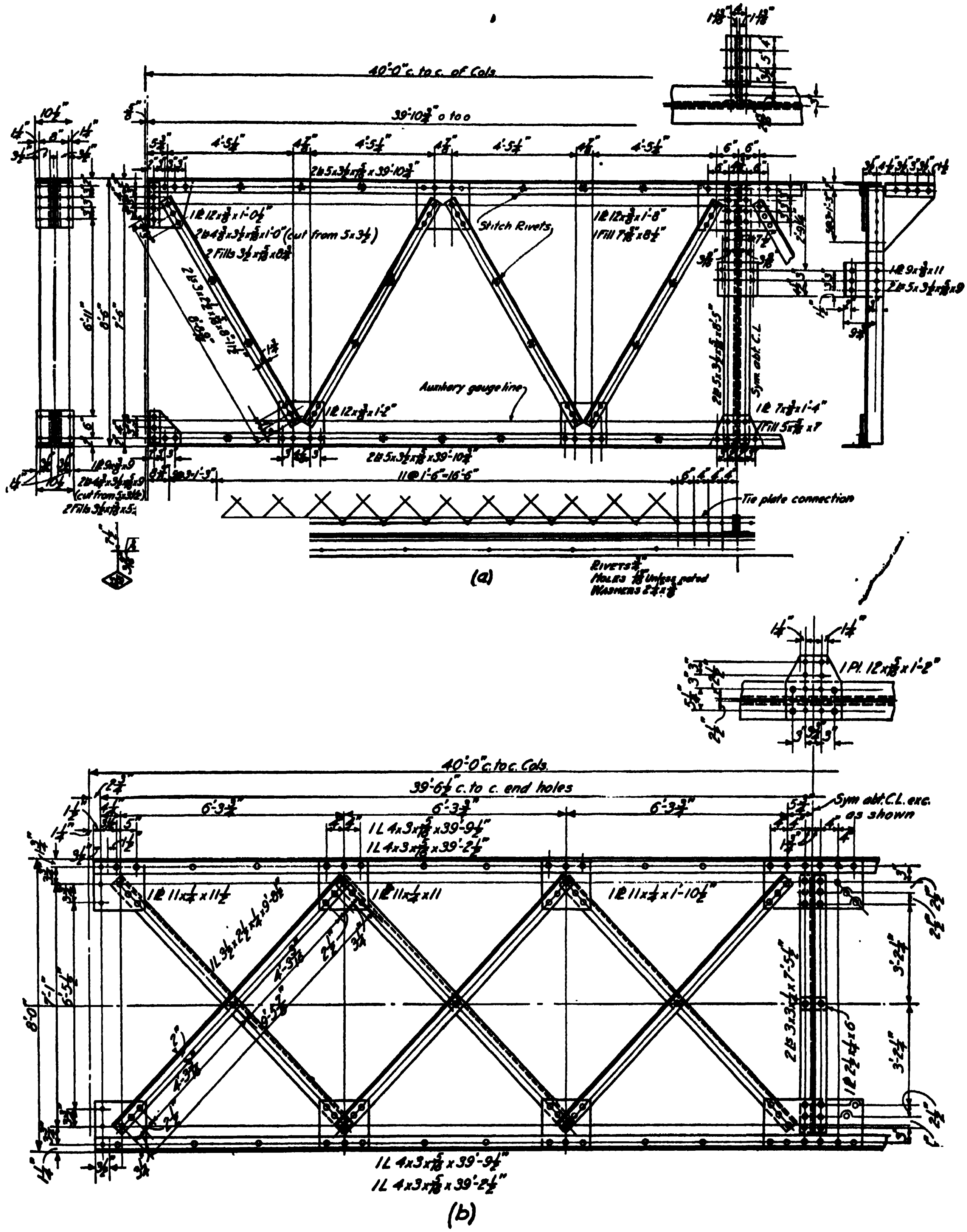


FIG. 152*

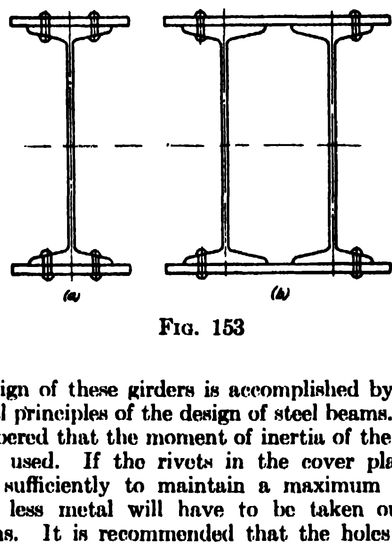
* Adapted from Bishop's "Structural Drafting," John Wiley and Sons, Inc., New York City.

elevation to make a lateral tie to the top flange of the crane girder by means of tie plates and lattice bars as illustrated in the figure. Another common use of these girders is in elevated railroad frames and lateral roof bracing.

Since a latticed girder is composed of members which constitute a triangular framing, the principles of truss design are involved (Part III). The common type approaches a Warren truss in nature although the depth is used in multiples of 6″, while the panels are made equal in length when possible. Although the working lines should theoretically be the gauge lines of the top and bottom chord angles, in order to eliminate secondary stresses, auxiliary working lines are usually employed for these light members, as shown in Fig. 152, in order to keep the gusset plates alike and thereby reduce the number of details and the number of templates. Stitch rivets should be used, as shown, spaced from 2′-0″ to 3′-0″ on centers.

86. Riveted Beam Girders.*

When the loads are excessive for the use of ordinary steel shapes, beams are sometimes reinforced by riveting plates to both the top and the bottom flanges, as shown in Fig. 153. That in (a) is the section used in practically all cases. Such a riveted beam girder is often more economical than two beams placed side by side (either with plates or with separators), and it admits repainting of the exposed surfaces.



(a) (b)

FIG. 153

The design of these girders is accomplished by employing the general principles of the design of steel beams. It should be remembered that the moment of inertia of the net section should be used. If the rivets in the cover plates can be staggered sufficiently to maintain a maximum net section (Art. 48), less metal will have to be taken out in these calculations. It is recommended that the holes in the top flange be deducted, as well as those in the bottom flange, for the reasons given in Art. 10† A trial section may be established by selecting a beam with a section modulus somewhat below that required, and adding plates of proportionate dimensions.‡

The cover plates need not extend the full length of the girder except when the girder is to carry a wall. In such a case it may be desirable to provide a level bedding surface for the wall. Otherwise, the points where the cover plates may be cut off, are established by the methods used for plate girders (Sect. 5D). The plates should extend a distance beyond the theoretical points of cut-off sufficient to develop the stress in them (usually taken as 1′-0″). The pitch of the rivets connecting the cover plates to the flanges of the beams may be calculated in the manner described in Arts. 64 and 65. It should be remembered that the plates add to the flexural resistance of the member only and hence no increase in web resistance is provided. Buckling and shearing stresses should therefore be particularly and carefully investigated. Stiffeners might be required for a given beam which was satisfactory in flexural resistance, but they are not ordinarily economical for rolled shapes, and a heavier beam should be used if the web is weak.

The relation of girder weight to carrying capacity can be best illustrated by an example as follows:

A 15 I 42.9 on a 20′-0″ span can carry a load of 31,400# uniformly distributed. A 15 I 42.9 riveted beam girder with 2-8″ × ½″ cover plates (weight 70.1#/ft.) on a 20′-0″ span can carry a load of 61,000# uniformly distributed (practically twice 31,400).

$$\frac{31{,}400}{42.9 \times 20} = 37.2 \text{ lbs. of load per lb. of beam.}$$

$$\frac{61{,}000}{70.1 \times 20} = 44.0 \text{ lbs. of load per lb. of beam.}$$

The increased efficiency results by adding the area of the plates at the extreme fibre where it is most effective in resisting flexure. The difference pointed out above should not be misinterpreted, because the cost of riveting the cover plates to the beam offsets the usual saving in weight, and architectural considerations of headroom and trim should necessitate these beams before they are resorted to.

Illustrative Prob. 86a. Calculate the maximum safe load per linear foot which a riveted beam girder composed of a 20 I 65.4 and two 10 × ⅝ plates can carry. ¾″ rivets. Span 20′-0″.

I of I-beam = 1169.5″⁴

I of 2 plates about their own axes neglected.

$A \cdot d^2$ for 2 plates (arm = 10 + $\frac{5}{16}$)

$2 \times 10 \times \frac{5}{8} \times \overline{10.31}^2$ = 1330

2499.5″⁴ gross

Grip of rivet = ¾ + ⅝ = 1⅜″

Area of 1 rivet hole = 1⅜ × ⅞ = 1.2□″

$A \cdot d^2$ for 4 rivet holes (arm = 10″ approximately)

$1.2 \times 4 \times (10)^2$ 480

2019.5″⁴ net

$$M_r = \frac{s \cdot I}{c} = \frac{16{,}000 \times 2019.5}{10.625} = 3{,}040{,}000''\#$$

$$M_e = 1.5\,w \cdot L^2 = 3{,}040{,}000 = 1.5\,w\,(20)^2$$

$$w = 5050\#/\text{ft. total}$$

$$\text{Wt. of gdr.} = \underline{110}$$

$$w = 4940\#/\text{ft. net.}$$

Prob. 86b. Design a riveted beam girder to carry a total load of 124,000# uniformly distributed on a 26′-0″ span. Use a 24″-81.4# I-beam as a basis. Determine the size of cover plates, their length and the pitch of rivets required.

* A selected line of riveted beam girders is given in the Carnegie Steel Company's "Pocket Companion" which have approximately twice the carrying capacity of the beams alone. The uniform loads they will carry on given spans, the increases in load carried for each $\frac{1}{16}$″ increase in the thickness of the flange plates added, the areas, gross section moduli, and weights per foot are tabulated.

† Some engineers deduct only the rivet holes in the bottom flanges in these calculations and assume the location of the neutral axis unchanged.

‡ The width of plate is governed by the projection beyond the rivets (Art. 65). The maximum thickness of plate is limited to the diameter of the rivet in order to avoid drilling holes.

PART II

DESIGN OF FLOOR CONSTRUCTION

CHAPTER 6

GENERAL CONSIDERATIONS

87. Live and Dead Loads Defined.

The design of floor construction is largely a practical application of the beam theory, and the design of beams (Part I), coupled with such limitations as the column design may impose (Part IV). The accuracy in stress computations is defeated unless the loads causing the stresses are calculated and estimated upon a scientific basis. The loads which a structure is designed for are commonly classed as dead loads and live loads. The former are developed by those portions of a building which constitute the physical enclosure and finish. Walls, permanent partitions, the floor frame, finish floors, roofs, and all other permanent construction entering into a building are examples of this class of load. The live loads include all loads which are indeterminate as to their exact locations at all times. Applied particularly to floor construction, the dead load is the weight of the flooring, fill, carrying floor and the ceiling attached to it, and the live load is any that is imposed upon the floor by the occupancy. Live loads often include those loads which are due to the possible relocation of partitions, but do not include partitions which form a definite enclosure, such as would occur around public spaces.

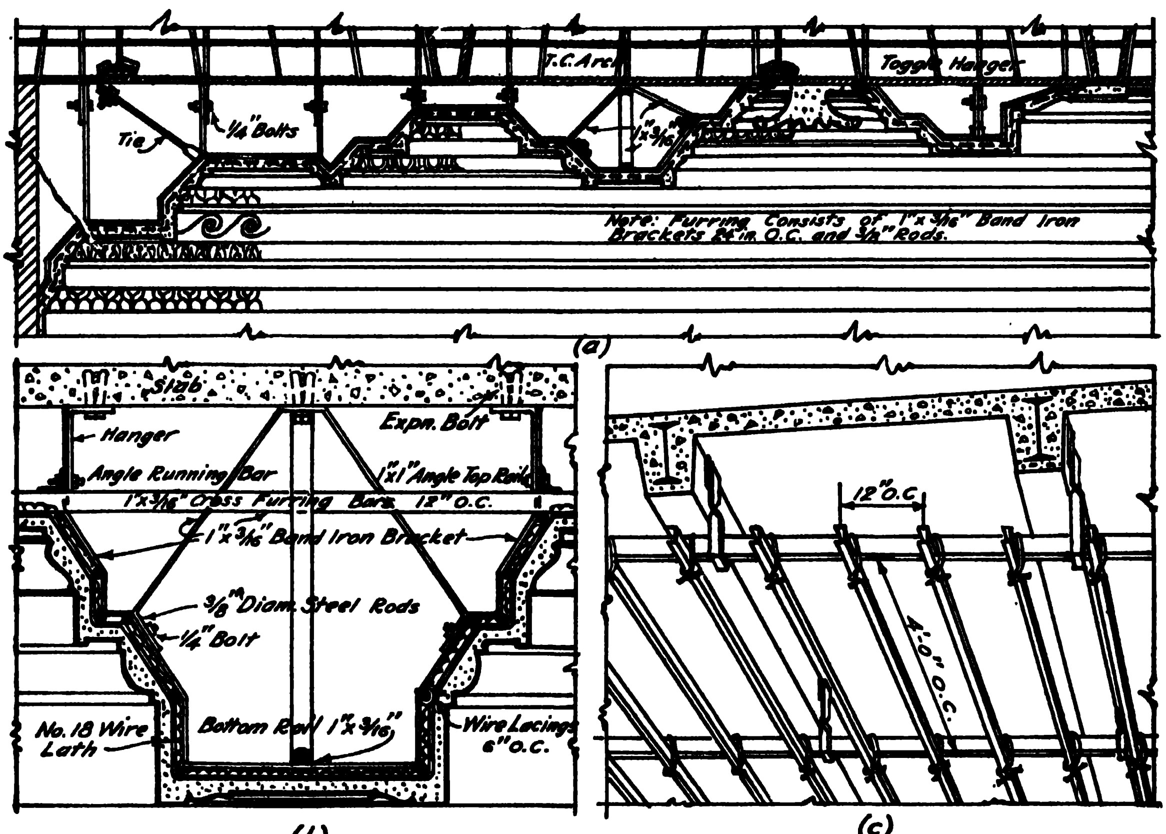


FIG. 154. SUSPENDED CEILINGS AND METAL LATH FURRING*

(a) furred beams and cornices (b) false beams (c) flat ceilings

* Courtesy of Wickwire Spencer Steel Corporation.

Prob. 87a. Is furniture in a house classed as live or dead load? What kind of load is plastering? What kind of load is a printing press?

88. Dead Loads.

As stated above, the weight of the floor construction itself is an important factor in the determination of the total load which a floor system carries. The necessary calculations involve the determination of the nature of the construction, as established by the architectural plans, and a reasonably careful estimate of the weights of the materials included in the floor finish, the fill if any, the definite floor structure, and the ceiling construction when it is attached to the carrying floor. Table 30 is useful in making the assumptions for such computations.

TABLE 30

WEIGHTS OF MATERIALS FOR FLOOR CONSTRUCTION

Finish Floors	
Floor Boarding (per inch of thickness)	3#/□′
Granolithic " "	12
Floor Tile " "	10
Stone Flaggings " "	14
Grout " "	13
Asphalt Mastic " "	12
Paving Brick " "	13
Wood Block " "	4
Linoleum (¼″ Std. Thickness)	1½
Rubber Tile (⅜″ Std. Thickness)	4
Fills	
Cinder Concrete (not tamped) (per inch of thickness)	6*
Screeds (Nailing strips)	2*
Sand (per inch of thickness)	8
Structural Floor	
Floor Boarding (per inch of thickness)	3
Plank Flooring " "	3½
Cinder Concrete (tamped) (per inch of thickness)	9
Stone Concrete "	12½
Terra Cotta Blocks "	1
Gypsum Blocks "	3
Steel Tile "	1
Structural Steel (per sq. ft.)	8–10
Reinforcing Steel " " "	4–6
Ceilings	
Plastered direct (2 coats)	5
Plaster on Wood Lath (direct)	6
" " " (suspended)	10
" Metal Lath (direct)	10
" " " (suspended)	15
Wood Ceiling Boards	2½
Stamped Steel Ceilings (including metal furring strips)	2

* Usually combined and estimated as 8#/□′.

When a suspended ceiling is required, the designer should make sure that he is making a sufficient allowance for it. If a plain, flat suspending ceiling is to be used, as shown in (c) in Fig. 154, then an allowance of 15#/□′, as given in Table 30, is amply sufficient, and some designers only allow 12#/□′. When more complicated ceilings are involved, the designer should make sure that 15#/□′ is ample, if the status of the architectural drawings permits him to do so. Some suspended ceilings, such as illustrated in (a) and (b), may weigh more than the usual amount.

The usual ceiling of this type consists of 1½″ × 3/16″ strap hangers (or 3/16″ φ rods), 4′-0″ o.c., looped around the steel beams or anchored into the slabs, supporting 1½″ cold rolled channels (or 3/16″ × ¾″ flat steel bars), 4′-0″ o.c. The latter serve as runners, and ¾″ cold rolled channels, usually 12″ o.c., running in the opposite direction, are attached to them with #16 gauge galvanized wire. The ¾″ channels serve as the furring to which the metal lath is clipped. The special ceilings may require extra hangers and bent bars, as shown in (a) and (b) in Fig. 154. Figure 186 also shows other details. Although a suspended ceiling is naturally more expensive than one applied directly to the construction, its advantages should be considered, as it has heat insulation value, particularly for the top story.

The allowance for the weight of the beams and girders in a particular system of framing is discussed later in connection with them. The weights of interior walls and the permanent partitions, which are a part of the dead load, as contrasted with movable partitions, are discussed in Part IV. In design work, the finish flooring and the floor fill (if any) are never included as a part of the strength of the floor, but their weights must be provided for, as for any other loads.

Prob. 88a. What is the dead load per square foot for a floor construction consisting of a 1″ wood finish flooring, 2⅝″ matched plank, and a suspended ceiling of metal lath and plaster?

89. Live Loads in General.

Live loads are specified as so many pounds per square foot of floor surface (#/□′) depending upon the use of the structure. Building codes state the amounts of minimum live load that must be used in any particular case, and if a structure is to be erected within the jurisdiction of a code, that ruling must be explicitly followed. In places where no code exists, that of a nearby city is specified or the recognized rules of good practice are followed.

By the nature of the problem, live loads must be estimated to approximate the extreme conditions which the floor will be subjected to. They must be conservative and yet not excessive, and must serve economy and safety. Live loads as given in various building codes are often inconsistent, even for a given kind of occupancy, as they do not have a common basis and in some cases are largely matters of tradition and ultra-conservatism. The minimum values imposed are in too many cases planned to cover unexpected and possibly unregu-

lated changes of occupancy. On the other hand, there are many instances of overloading, particularly in storage buildings, due to the lack of occasional inspection and the provision for the enforcement of wise decisions following such inspections. In place of completely listing the usual classes of occupancy, many codes leave the minimum requirements to the building commissioner, which is distasteful to him, and makes irregularities possible. As a consequence, there has been considerable discussion relative to the drafting of standardized building laws which could be easily adopted as municipal ordinances. The most recent example of the movement along these lines is that of the investigation by the United States Senate Committee on Reconstruction and Production, appointed in 1920. The following excerpts from the reports of that committee are significant:

> "The building codes of the country have not been developed upon scientific data, but rather on compromises; they are not uniform in principle and in many instances involve an additional cost of construction without assuring more useful or more durable buildings."*
>
> "A study of these codes and experience under them would be of great service in preparing the material for the drafting of a building code which would be as nearly uniform as the varying conditions in the different cities would permit; . . . new drafts of codes could be prepared in the light of the collected experience of the whole country, and not as a result of purely local consideration. . . . "A great saving in building throughout the country could be secured by careful study of building construction and standardization of building materials similar to the work done by the Bureau of Standards in other lines."†

Secretary Hoover, of the Department of Commerce, recognizing the necessity for some central coördinating body to standardize, as far as possible, the building laws of the country, organized the Building Code Committee for that purpose. The committee is a part of the newly created division of building and housing, which has under way a broad program of investigation into the causes of the sluggishness in the building industry and the possible remedies which may suggest themselves. This committee, as a part of its work, has published a report on "Minimum Live Loads Allowable for Use in Design of Buildings." This report presents load requirements recommended for general adoption with the object of preserving safety, stimulating uniformity of requirements, and effecting conservation of materials and labor. Some of the loads given in the report are lighter than those corresponding in many codes, but each is reasonably conservative in view of the large amount of data studied. The loads may be used for the purpose intended, if a system of control is maintained after the structure is erected. They may be applied to portions of buildings rather than to buildings as a whole. Special cases should be designed for the loads which the occupancy suggests, such as library stack rooms, laboratories, and so on. In view of a building being sold, or let for another purpose involving heavier loads, the committee suggests designing some bays for heavier loads than others so that the new occupancy could be distributed according to the placarded loadings.

* Preliminary report.

† Senate Report No. 829, p. 57.

Live loads may be divided into two classes as suggested by the Building Code Committee, namely:

(1) human occupancy, and

(2) industrial or commercial occupancy.

90. Live Loads for Human Occupancy.

In the study of the loads on floors in rooms of habitation, or in rooms with fixed seats, considerable data have been compiled. In residences, the loads from furniture do not average over 10#/□′ and the heaviest loads discovered by investigators are those of pianos, which may approach 55#/□′ of horizontal projection, and bookcases, which may reach 170# per linear foot. These are, however, distributed in such a way as to bring the equivalent uniform load below that usually specified. In hotel rooms, the furniture averages about 4#/□′‡. In school classrooms, the average load including both furniture and pupils is about 10 to 12#/□′ under normal conditions, but this load may approach 30#/□′ under extremely crowded conditions.§ In hospital wards, the load averages from 7 to 9#/□′ including the furniture and patients.||

In general, from the reported observations of several reliable investigators the loads from furniture seldom exceed 20#/□′ in the majority of cases, but the loads induced by crowds in the types of rooms discussed above may average 40#/□′.

SPECIFICATION CLAUSE¶

Dwelling Room Floors and Fixed Seats 1. For rooms of private dwellings, hospital rooms and wards, guest rooms in hotels, lodging and tenement houses, and for similar occupancies, the minimum live load shall be taken as 40 pounds per square foot uniformly distributed, except that where floors of one and two family dwellings are of monolithic type, or of solid or ribbed slabs, the live load may be taken as 30 pounds per square foot.

‡ Investigations by the Hotels Statler Co.

§ See article in Engineering News Record, May 6, 1920, relative to loading tests by the Milwaukee Board of Education. Also see article in the American Architect, April 11, 1923, relative to the investigations of Norman M. Steinman.

|| Based upon measurements obtained at the New York State Hospitals for the Insane at Rochester and Brooklyn.

¶ Report of the Department of Commerce Building Code Committee on "Minimum Live Loads Allowable for Use in Design of Buildings," issued in 1924.

The above specification, which is also included in the committee's final report on "Recommended Minimum Requirements for Small Dwelling Construction," received practically unanimous approval from those to whom a preliminary report was sent for critical discussion. The smaller load is allowed for monolithic floors because of the more coherent distribution of load, the greater proportion of dead load to live load, and the inherent rigidity of such construction.

The live loads in **office buildings** may reach a higher value because the furniture often is of a heavier type, especially when safes, sectional filing cases, card filing cabinets, and the like, are considered. In the majority of cases, the larger proportion of the load is confined to a zone approximately 3′-0″ wide around the walls and partitions and hence is carried directly to some unit of the structural frame. The arrangement of the office may concentrate the furniture somewhere within the central zones of the floor panels, and this condition actually exists at times when cleaning or moving is in progress. When a building is properly designed to obtain sufficient light and ventilation, the maximum condition probably is to have three fully loaded offices or storerooms tributary to one column. At any rate there is an indefinite factor in the live loads which movable objects impose upon a floor system. It is therefore imperative that the engineer analyze the situation so as to determine the economic maximum which may exist due to the ordinary use of the building.

The average load from the typical office furniture is from 7 to 9#/□′, while that imposed by the human occupancy is from 1.5 to 2.5#/□′. For an extreme case, such as would exist in a large stenographic office, the total load may approach 30#/□′.*

SPECIFICATION CLAUSE†

Office Floors 2. For floors for office purposes and for rooms with fixed seats, as in churches, school classrooms, reading rooms, museums, art galleries, and theaters the minimum live load shall be taken as 50 pounds per square foot uniformly distributed. Provision shall be made, however, in designing office floors for a load of 2000 pounds placed upon any space 2½ feet square wherever this load upon an otherwise unloaded floor would produce stresses greater than the 50-pound distributed load.

In public places, or other spaces where at times crowds assemble, it has been shown that it is possible to obtain a live load of 140#/□′ by using unusual experimental methods. Such a load could not exist for any considerable time because of the discomfort of the people. Consequently it is unreasonable to design for such a load and a lesser value is commonly allowed. When such a condition does exist, the factor of safety will provide protection but in no case should the stresses developed exceed the elastic limit of the material. A general check should be made for main carrying members to determine their resistance to the maximum condition.

SPECIFICATION CLAUSE†

Floors in Public Spaces 3. For aisles, corridors, lobbies, public spaces in hotels and public buildings, banquet rooms, assembly halls without fixed seats, grandstands, theater stages, gymnasiums, stairways, fire escapes or exit passageways, and other spaces where crowds of people are likely to assemble, the minimum live load shall be taken as 100 pounds per square foot uniformly distributed. This requirement shall not apply, however, to such spaces in private dwellings, for which the minimum live load shall be taken as in paragraph 1 of this section.

Prob. 90a. What live loads per square foot should be used for a hospital in the wards, private rooms, stairways, and fire escapes?

91. Live Loads for Industrial or Commercial Occupancy.

When live loads are considered for buildings other than those for human occupancy, more factors enter into the problem, due to the many kinds and weights of machinery and manufactured products. In any case, a minimum live load should be established, but in particular instances, the value may have to be increased to correspond with the actual conditions. When heavy machinery is imposed upon a floor it must be remembered that additional practically vacant space must be provided for the ease of operating the machines. This relieves the structural parts immediately under the machines to some extent. The lay out of these special cases should be reasonably well fixed before the design is undertaken. When this forethought is given to the problem many of the concentrations may be carried directly to the girders and columns without becoming a factor at all in the general live load considerations.

SPECIFICATION CLAUSE†

Floors for Storage and Manufacture In designing floors used for industrial or commercial purposes, or purposes other than previously mentioned, the live load shall be assumed as the maximum caused by the use which the building or part of the building is to serve. The following loads shall be taken as the minimum live loads permissible for the occupancies listed, and loads at least equal shall be assumed for uses similar in nature to those listed in this section.

* See article in "American Architect and Architectural Review," Jan. 3, 1923. Also see article in the Engineering News Record, March 29, 1923, entitled "Live Loads in Office Buildings," based upon data obtained by Mr. C. T. Coley, manager of the Equitable Building, New York City.

† Report of the Department of Commerce Building Code Committee on "Minimum Live Loads Allowable for Use in Design of Buildings," issued in 1924.

Floors used for —	Minimum live load (lbs. per sq. ft.)
Storage purposes (general)	100
Storage purposes (special)	100
Manufacturing (light)	75
Printing plants	100
Wholesale stores (light merchandise)	100
Retail salesrooms (light merchandise)	75
Stables	75
Garages —	
All types of vehicles	100
Passenger cars only	80
Sidewalks	250
(or 8000 pounds concentrated, whichever gives the larger moment or shear).	

It should be carefully noted that the above are **minimum values,** and in many cases, much larger values must be used.

For manufacturing buildings, the following table shows some interesting data.

TABLE 31

DATA ON FLOOR LOADS IN MANUFACTURING BUILDINGS*

Occupancy:	Observed loads (lbs. per sq. ft.)
1. Automobile plants —	
Machine shop floors	60, 75, 80, 95, 100, 110, 115, 120, 125, 200.
Body building	15–35.
Motor assembly	110.
Car assembly	60.
Ovens	150.
Furnaces	300.
Storage of parts	60, 65, 70, 75, 80, 85, 90, 110, 140, 145, 165, 195, 225, 235, 345.
Storage of bodies	14, 27, 32, 33, 41, 50.
2. Automobile tire plants —	
Vulcanizers	175.
Dryers	75.
3. Textile mills —	
Cardrooms	80.
Other departments	50.
4. Machine shops —	
Light work	7 bays at 30 pounds, 2 at 35, 4 at 40, 3 at 45, 9 at 50, 4 at 55, 5 at 60, 2 at 70, 5 at 75, 2 at 85, 3 at 90, 1 at 95, 2 at 100.
Heavy work	150–175.
5. Garment factories	100.
6. Perfumery works	40, 65, 75, 85, 105, 120, 150.
7. Printing and binding —	
Heavy pressrooms	250–400.
Light presses	175.
Composing rooms	75–85.
Linotype rooms	75–85
Type cases, closely packed	250.
Stereotype rooms	200–250.

The live loads for warehouse construction should be even more carefully decided upon, because of the many variations of storage. Table 32 is useful in this connection.

TABLE 32

DATA ON FLOOR LOADS IN STORAGE BUILDINGS*

Class of Commodity	Maximum Probable Weight per Cubic Foot of Storage Space† (Lbs.)	Maximum Probable Weight per Square Foot of Storage Space‡ (Lbs.)	Observed Loads per Square Foot in Storage Buildings (Lbs.)
Acids	55	440	
Agricultural machinery	55	440	
Asbestos	50	400	200
Automobiles, crated	13	104	
Automobile parts	40	320	50, 58, 60, 75, 80, 90, 110, 140, 150, 165, 200, 225, 235, 345, 700
Automobile tires	30	240	90, 100, 171
Automobiles, uncrated	8	64	
Baggage:			
Empty	6	48	
Packed	20	160	
Beverages	40	320	
Bricks:			
Building	45	360	
Fire-clay	75	600	
Cable and wire	75	600	220
Carpets and rugs	30	240	
Cement	65	520	
Cereals	45	360	256, 200
Chain	100	800	1200
Chemicals	50	400	370, 600
Clocks and watches	40	320	
Cocoa	35	280	210, 450
Cotton:			
American .... bales	30	240	
Foreign .... do	40	320	180
Cotton goods	45	360	
Cutlery	45	360	
Electrical goods and machinery	40	320	
Extracts	60	480	
Flour and meal	45	360	200, 300–400, 350
Fruits, dried or canned	50	400	350, 450, 340, 315, 300–400, 400
Fruits, fresh	35	280	250
Furniture	20	160	250, 60–85, 100
Guns and ammunition	65	520	
Gypsum	50	400	
Hardware, small	110	880	250
Hides, green	55	440	
Hemp, jute and other fibres	35	280	220
Leather and leather goods	40	320	175, 210, 260, 85–300
Machinery, light	20	160	150
Meat and meat products	45	360	410, 370, 340
Milk, condensed	50	400	370, 305, 365, 360, 385
Nonferrous metals, bulk	250	2000	
Oils and greases	45	360	240
Paints	90	720	
Paper and books	50	400	270, 395, 340, 300, 275, 500, 200
Photographic supplies	40	320	
Plumbing:			
Fixtures	30	240	
Supplies	55	440	75
Potash	55	440	300, 330
Rope, fibre	30	240	
Rubber, crude	50	400	250, 310, 400
Shafting, steel	125	1000	435, 650
Silk and silk goods	45	360	
Soaps	50	400	
Steel, bulk	225	1800	
Sugars, sirups, and candies	50	400	200, 350, 325, 350, 300–400
Tiles	50	400	
Tobacco bales	35	280	180
Tobacco, hogsheads and bbls.	28	224	
Toilet articles, miscellaneous	35	280	
Tools, small, metal	75	600	100
Trucks	22	176	250
Varnishes	55	440	
Vegetables, canned or dried	45	360	285, 250, 300–400, 400
Woods, bulk	45	360	
Wool and woolen goods	50	400	245, 250, 330

* Report of the Department of Commerce Building Code Committee on "Minimum Live Loads Allowable for Use in Design of Buildings," issued in 1924.

† The maximum probable weight per cubic foot of storage space is based upon careful study of data compiled by the U. S. Shipping Board for use of the American Expeditionary Force and published in "Stowage Factors for Ship Cargoes," obtainable from the Superintendent of Documents, Government Printing Office, Washington, D. C., at 35 cents per copy.

‡ The maximum probable weight per cubic foot of storage space is based upon an 8′-0″ depth of stored material, which is considered as a fair average basis, having in mind the use of modern elevating machinery for piling packages to full-story height in warehouses, the necessary clearance for handling goods, or for effective sprinkler action.

Prob. 91a. What live loads should be used, in the design of a plant manufacturing automobile tires, for the rooms in which light machines are to be operated, storage rooms for tires, storage rooms for the chemicals, rooms for the vulcanizers, those for the dryers, and the office portion?

92. Allowance for Movable Partition Loads.

An estimate of a uniformly distributed load which will be equivalent to the effect of relocating partitions, is difficult to make. In residential buildings and in apartment houses of modern design, there is little probability of the partitions being rearranged. In storage buildings and those planned for heavy manufacturing, the question is also not important, as the weight of the partitions is generally small compared with the heavy live loads required. In office buildings, public buildings, and those planned for light manufacturing, however, relocation of partitions is quite probable. From a study of this situation* it was found that the distributed partition weight might vary from 25# to 30#/□′ should all the partitions be included. For the types of buildings referred to above, designers usually make an allowance of from 15# to 20#/□′ in the tabulation of the floor load to provide for random partitions.

SPECIFICATION CLAUSE†

Movable Partitions Floors in office and public buildings, and in other buildings subject to shifting of partitions without reference to arrangement of floor beams or girders shall be designed to support, in addition to other loads, a single partition of the type used in the building, placed in any possible position.

ALTERNATE CLAUSES

Beams and Girders All beams and girders shall be designed to sustain in addition to other loads a partition of not less than 300 pounds per linear foot less the specified live load per square foot.

Floor Elements Arches, slabs, joists, and other direct load carrying elements shall be designed to sustain in addition to the other loads, a minimum concentrated cross partition load of 300 pounds for each linear foot of width of the strip on center to center spacing of joists, applied at the point of maximum moment.

The variations involved in the design of floor systems where movable partitions must be provided for, naturally impose upon the engineer a study of each particular case. The above specification is an attempt to provide minimum requirements. In the case of beams and girders, the full partition load is included because of the natural lines of subdivision which they afford. For this reason it is customary to provide beams under partitions when possible and consistent with architectural effect, should the partition be later removed. If a partition is changed from one location to another, an attempt is usually made to relocate it over an existing beam, if possible. It is therefore advisable to design these members for such a contingency. The object of deducting the live load per square foot from the weight of the partition per linear foot should be obvious when one considers the fact that two objects can not occupy the same space at one time, and furthermore that furniture or occupants must remain at least 3″ away from the plaster line.

The direct load carrying elements which span between the beams present a different condition. It is entirely possible for a partition to impose a load of about 300 pounds per linear foot upon such an element, especially if it runs parallel to the span of the element. It is, however, improbable that this single element will alone resist the partition load, and the authors believe that under the usual conditions the adjoining strips will assist in carrying the load. It is wise to make all elements sufficiently strong to carry a minimum of 300 pounds per foot of width as specified, concentrated at the center of the span of the element in order to provide for transverse partition loads.

A very excellent method is to study the layout with the object in mind of determining the actual maximum number of linear feet of partition which a definite area might have and still be useful. The total weight of these partitions may then be distributed over the area and a check made of the load. The concentrations mentioned above should be provided for, in addition. The partitions coming directly on main beams or girders may be omitted in this general study but should be included in the design of the beams which directly support them.

93. Reduction of Live Load.

When a beam or girder carries a considerable portion of a floor, it is reasonable to expect that the **probability** of the **full live load existing** simultaneously **on every square foot of floor** surface contributing to the load on the member, **is remote.** This, of course, is more probable in certain types of buildings than in others. Many building codes take this phase of loading into consideration for specified types of occupancy.

SPECIFICATION CLAUSES‡

Areas Less than 100□′ Every plank, slab and arch, and every floor beam carrying one hundred square feet of floor or less, shall be of sufficient strength to bear safely the combined dead and live load supported by it, but the floor live loads may be reduced for other parts of the structure as follows: —

Areas 100 to 200□′ In all buildings except armories, garages, gymnasiums, storage buildings, wholesale stores, and assembly halls, for all flat slabs of over one hundred square feet area, reinforced in two or more directions, and for all floor beams,

* See first footnote, p. 129.

† Report of the Department of Commerce Building Code Committee on "Minimum Live Loads Allowable for Usé in Design of Buildings," issued in 1924.

‡ The Building Law of the City of Boston.

girders, or trusses carrying over one hundred square feet of floor, ten per cent reduction.

Areas 200 to 300□' For the same, but carrying over two hundred square feet of floor, fifteen per cent reduction.

Areas Exceeding 300□' For the same, but carrying over three hundred square feet of floor, twenty-five per cent reduction.

More than One Floor These reductions shall not be made if the member carries more than one floor and therefore has its live load reduced. . . .

Garages In public garages, for all flat slabs of over three hundred square feet area reinforced in more than one direction, and for all floor beams, girders, and trusses carrying over three hundred square feet of floor, . . . twenty-five per cent reduction."

ALTERNATE SPECIFICATION CLAUSE*

Floors, joists and beams shall be designed for the full dead and live loads. Floor girders shall be designed for the full dead and not less than eighty-five per cent of the live load.

Prob. 93a. If a girder carries 180□' of floor and the loads per square foot are 150#/□' L.L. and 100#/□' D.L. in a machine shop, what superimposed total load may the girder be designed to carry?

Prob. 93b. If the load data of Prob. 93*a* are used and the girder carries 280□' of floor in a garage, what superimposed total load should the girder be designed to carry?

94. Increase of Usual Values for Live Load.

Impact effects of moving loads in buildings can usually be neglected. In special cases of industrial occupancy, the dynamic effect of moving machinery must be considered in addition to its weight, because of the vibration induced. In some instances, this is provided for by the use of more rigid floor construction or extra bracing. Some designers prefer to increase the live load.

SPECIFICATION CLAUSE

Impact For structures carrying machinery, such as cranes, conveyors, printing presses, etc., at least 25 per cent shall be deducted from the allowable stresses to provide for effect of impact and vibrations.

In certain cases, such as for factories, lofts or warehouses, it is wise to provide for unusual load conditions, as follows:

SPECIFICATION CLAUSE

Any floor beam or girder shall be sufficient to carry a live load of 4000# concentrated at the center of its span.

95. Control of Floor Loadings.

As a means of obtaining adequate supervision of floor loadings, some building departments of city governments issue occupancy permits. If a floor is strong enough for its intended use, there is no need of providing an additional surplus so that considerable economy can be effected by using loads which are reasonably low. However, the additional cost of a heavier floor and supporting members may be returned many times over in superior adaptability of the building, so that a study of this situation is always advisable (Art. 98). Zoning ordinances are now being adopted in many cities, and these are an aid in establishing floor loads. Many building departments require that for mercantile buildings, the allowable load should be conspicuously posted in each story by the use of permanent floor-load placards.

SPECIFICATION CLAUSES†

Floor Capacities 1. *Estimate of floor capacity.* In every building now existing or hereafter erected, occupied wholly or in part as a business building, in which heavy materials are kept or stored, or machinery is introduced, the weight that each floor will safely sustain shall be estimated by the owner or occupant, or by a competent person employed by the owner or occupant. Such estimate shall be filed with the superintendent of buildings, properly verified by the person making the same in such manner as such superintendent may direct, and shall give full information on which the estimate is based. When such estimate is found to be satisfactory and correct, the superintendent of buildings shall approve the same. If the superintendent of buildings shall have cause to doubt the correctness of said estimate, he is empowered to revise and correct the same and for the purpose of such revision the officers and employees of the bureau of buildings may enter any building and remove so much of any floor or other portion thereof as may be required to make necessary measurements and examination. Any expense necessarily incurred in removing any floor or other portion of any building for the purpose of making any examination herein provided for shall be paid by the comptroller, upon the requisition of the superintendent of buildings, out of the fund paid over to him under the provisions of § 639 of this chapter. Such expenses shall be a charge against the person or persons by whom or on whose behalf said estimate was made, provided such examination proves the floors of insufficient strength to carry with safety the loads found upon them when such examination was made; and shall be collected in an action to be brought by the corporation counsel against said person or persons, and the sum so collected shall be paid over to the comptroller, to be deposited in said fund in reimbursement of the amount paid as aforesaid.

2. *Posting floor capacities.* Before any building hereafter erected is occupied, in whole or in part, as a business building, and before any building already erected but not heretofore occupied as a business building is occupied or used, in whole or in part, for such purpose, the safe live load for each floor as approved by the superintendent of buildings shall be posted in a conspicuous place in the story to which it

* Building Ordinance of the City of Chicago.

† The Code of Ordinances of the City of New York.

relates. When the safe live load for any existing floor, ascertained as hereinbefore provided, has been approved by the superintendent of buildings, the owner or occupant shall post such approved live load in a conspicuous place or places on each story occupied for any of the purposes indicated in this section.

3. *Loading of floors.* No person shall place, or cause or permit to be placed, on any floor of any building any greater load than the approved safe load.

4. *Safes.* No safe shall be placed on a stair landing or in a stair hall, nor shall its weight be carried by any beam which also carries the floor of any landing or stair hall.

The following* is a sample of such a floor load placard:

THIS FLOOR WILL SAFELY SUSTAIN

150

POUNDS PER SQUARE FOOT
UNIFORMLY DISTRIBUTED

IT IS UNLAWFUL TO PLACE ANY
GREATER LOAD ON THIS FLOOR
OFFENDERS ARE SUBJECT TO PROSECUTION

OCTOBER
1924

JOHN DOE
BUILDING INSPECTOR

Permission usually must be obtained from the building department to use a building for a heavier load than posted upon the placard.

96. Floor Tests.

A common building code specification, when a newly proposed system of fire resisting floor construction is not susceptible of analysis and computations according to the usual methods of design, is to require a test of a sample panel.

SPECIFICATION CLAUSE†

When the strength of any floor construction cannot be determined by the methods prescribed in this section or by the application of accepted engineering formulas, the safe uniformly distributed carrying capacity shall be taken as one sixth of the total load causing failure to a full-sized construction with the load applied at two points, each at one third of the span from the ends of the span.

Another instance of floor tests is when there is doubt as to the quality of the materials used or of the workmanship, or of both, in an accepted type of floor construction already in place.

SPECIFICATION CLAUSE†

The commissioner may order loading tests to be made, at the expense of the owner, on any structure or part thereof, at such time and in such manner as will satisfactorily demonstrate to him that the unit stresses in any materials do not exceed those permitted under this act. Concrete construction shall be capable of bearing a live and dead load equivalent to twice that for which it was designed without causing permanent deformation.

The common method of loading for the first kind of test is that of employing two equal concentrated loads at the third-points of the span. This is done on account of the difficulty and cost of providing a uniform load which will cause failure, and also because the arching effect in the loading materials makes the results in a uniform loading test unreliable. Furthermore, a uniform moment exists between the loads. In the second kind of test the floor is subjected to a uniform load equal to twice the live and dead load designed for, which it must sustain for 24 hours without injury to the floor or any permanent deflection. This corresponds approximately to a factor of safety of 4, based upon the total dead and live loads. Both kinds of the tests are usually made on a section of floor not less than 4′-0″ wide and on a full span length. Figure 155 shows the moment and shear coefficients for the two cases for two equal spans, only one of which is loaded. These are obtained by an application of the principles of continuous beams. In the case of the test of a new type of floor construction to ultimate failure, the allowable working load is generally taken as $\frac{1}{6}$th the total load (one-half is applied at each third-point) which caused failure. This corresponds to a factor of safety of approximately 8 on the basis of a uniformly distributed load. This higher value is desirable for a combination of materials the strength of which is not well established. This relation may be developed in the following manner:

Let W = the sum of the two concentrated loads applied at the third-points of the span,
M = the external bending moment caused by such loading,
W_u = the equivalent uniform load,
M_u = the external bending moment caused by W_u,
L = the span in feet,
B = the breadth of the panel under test, and
W_s = the allowable live load on the floor panel.

For a simple span,

$$M_u = \frac{W_u \cdot L}{8} \quad \text{and} \quad M = \frac{W}{2} \cdot \frac{L}{3} = \frac{W \cdot L}{6}$$

$$\frac{M}{M_u} = \frac{1}{6} \div \frac{1}{8} = \frac{4}{3} = 1.33$$

* Report of the Department of Commerce Building Code Committee on "Minimum Live Loads Allowable for Use in Design of Buildings."

† The Building Law of the City of Boston.

$$\text{Hence } \frac{W_u}{W} = \frac{4}{3}, \quad \text{or} \quad W = \frac{3\,W_u}{4}.$$

$$W_s = \frac{W}{6}, \text{ or } W_s = \frac{1}{6} \times \frac{3\,W_u}{4} = \frac{W_u}{8}.$$

For a restrained span, as in Fig. 155 (*a*), when the anchor span has one end free as is preferable,

$$M = +\frac{1}{7.2} W \cdot L.$$

$$\text{From } (b), \quad M_u = +\frac{1}{10.5} W_u \cdot L.$$

$$\text{Hence} \quad \frac{M}{M_u} = \frac{1}{7.2} \div \frac{1}{10.5} = 1.46, \quad \text{or}$$

$$\frac{W_u}{W} = 1.46, \quad \text{and} \quad W = \frac{W_u}{1.46}.$$

$$\text{As before} \quad W_s = \frac{W}{6} \quad \text{or} \quad W_s = \frac{1}{6} \times \frac{W_u}{1.46} = \frac{W_u}{8.76}.$$

In either case, the allowable load in pounds per square foot is W divided by the area under test and a factor of safety of 6, or

$$w_0 = \frac{W}{6\,B \cdot L}.$$

The same relations may also be shown to be true by comparing the corresponding coefficients for the case with both ends of the anchor span fixed, and again for the negative moments in both cases.

In fire-resisting construction, strength to carry loads is of course essential, but sufficient strength in case of fire is also required. Incombustible materials are not the only requirement. They should possess fire-resistance and the construction should be a poor conductor of heat. A low coefficient of expansion is also desirable, both in the action of a fire, and the subsequent reactions when hose streams strike the materials. Consequently, fire, load, and water tests are often specified, especially when new types of construction are proposed for use.

SPECIFICATION CLAUSE†

Test of Floor Fillings (*a*) Fire Tests: In testing the fireproof qualities of any floor filling, at least one panel of the proposed maximum span, carrying a live load of at least 150 pounds per square foot, shall be subjected to a fire continuous for 4 hours at an average temperature of 1700 degrees F., followed by the application for not less than 10 minutes of a hose stream from a 1⅛ in. nozzle at 60 pounds pressure, without appreciable deterioration, or the passage of flame through the floor during the test.

Structural steel commences to lose some of its strength at a temperature around 500° F., and at about 1000° F., it loses about 70% of its strength. Temperatures in severe fires often exceed these values, so that exposed structural steel is subject to considerable distortion in such cases, and even collapse.

Prob. 96a. If a floor panel 4′-0″ × 12′-0″ is tested to failure and carries a total ultimate load of 102,000#, what safe load per sq. ft. should be allowed?

97. Classes of Construction.

Buildings are commonly divided into three classes, namely first, second, and third. Some codes further classify them according to the occupancy or fire-resistance. The following are typical examples of the requirements and limitations of each class:

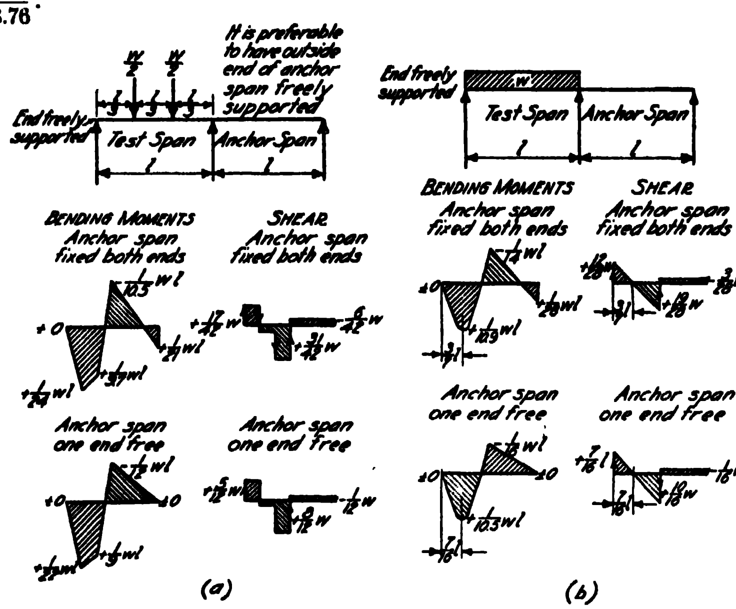


FIG. 155*

SPECIFICATION CLAUSES‡

In this act the following terms shall have the meanings respectively assigned to them as follows:

First Class Building A first class building shall consist of fireproof material throughout, with floors constructed of iron, steel or reinforced concrete beams, filled in between with terra cotta or other masonry arches or with concrete or reinforced concrete slabs; wood may be used only for under and upper floors, windows and door frames, sashes, doors, interior finish, hand rails for stairs, necessary sleepers bedded in the cement, and for isolated furrings bedded in mortar. There shall be no air space between the top of any floor arches and the floor boarding.

Second Class Building All buildings not of the first class, the external and party walls of which are of brick, stone, iron, steel, concrete, reinforced concrete, concrete blocks, or other equally substantial and fireproof material.

* Based upon the figure shown in the Building Code of the National Board of Fire Underwriters.

† Laws and Regulations of the Building Code of the City of New York, as amended to May 1, 1922, covering Fireproof Construction, Sec. 354, Art. 5.

‡ The Building Law of the City of Boston.

Third Class Building

A wooden frame building.

Composite Building

A building partly of second class and partly of third class construction. Composite buildings may be built under the same restrictions as, and need comply only with the requirements for, third class buildings as to fire protection and exterior finish.

Every building hereafter erected more than seventy-five feet in height, or hereafter increased in height to more than seventy-five feet, shall be a first class building. Every second class building hereafter erected, and more than four stories in height, and any second class building now in existence and increased in height to more than four stories shall have the first floor and the basement and cellar stories of first class construction. Every hotel, tenement house and lodging house hereafter erected covering more than three thousand five hundred square feet, or more than five stories in height, shall be a first class building; and every building altered or enlarged and occupied or to be occupied as a hotel, tenement or lodging house to be in excess of sixty feet in height, or in excess of three thousand five hundred square feet in superficial area, or in excess of five stories in height, shall be a first class building. Every building hereafter erected within the building limits to be occupied as a permanent schoolhouse shall be a first class building. Every building hereafter erected as a theatre, and every building hereafter altered to be occupied as a theatre, shall be a first class building. Every building hereafter erected for, altered to or converted to use as a moving picture house shall be a first class building. All other buildings may be of second or third class construction.

Except as is otherwise provided herein, new buildings adapted for habitations, and not more than five stories in height, may be erected of second class construction, but no such building shall exceed three thousand five hundred square feet in superficial area or sixty feet in height. The first story or basement, or both the first story and basement, in such buildings, so constructed, remodelled, or enlarged may be used for mercantile purposes, provided that the first floor and the basement and cellar stories shall be of first class construction, and any stairway leading from the first floor to the basement or from the basement to the cellar shall be enclosed in masonry walls not less than eight inches thick or with two inch solid metal and plaster partition, with self-closing fireproof doors at the top and bottom of the stairway.

SPECIFICATION CLAUSES*

Classification of Buildings

Occupancy.
Construction.
When buildings are required to be fireproof.
When buildings may be non-fireproof.
One-story special buildings.

Occupancy

1. *Classes designated.* For the purposes of this chapter, all buildings or structures shall be classified, with respect to occupancy and use, as public buildings, residence buildings and business buildings, as hereinafter specified and defined.

2. *Public buildings.* Public buildings are buildings or parts of buildings in which persons congregate for civic, political, educational, religious or recreational purposes, or in which persons are harbored to receive medical, charitable or other care or treatment, or in which persons are held or detained by reason of public or civic duty, or for correctional purposes, including among others, court houses, schools, colleges, libraries, museums, exhibition buildings, lecture halls, churches, assembly halls, lodge rooms, dance halls, theatres, bath houses, hospitals, asylums, armories, fire houses, police stations, jails and passenger depots.

3. *Residence buildings.* Residence buildings are buildings or parts of buildings in which sleeping accommodations are provided, except such as may for other reasons be classed as public buildings, including among others, dwellings, tenement houses, hotels, lodging houses, dormitories, convents, and studios and club houses having sleeping accommodations.

4. *Business buildings.* Business buildings are buildings or parts of buildings, which are not public buildings or residence buildings, including among others, office buildings, stores, markets, restaurants, warehouses, freight depots, car barns, stables, garages, factories, laboratories, smoke houses, grain elevators and coal pockets.

5. *Doubtful classification.* In case any building is not specifically provided for, or where there is any uncertainty as to its classification, its status shall be fixed by a rule promulgated by the superintendent of buildings.

6. *Mixed occupancy.* In case a building is occupied or used for different purposes in different parts, the provisions of this chapter applying to each class of occupancy shall apply to such parts of the building as come within that class; and if there should be conflicting provisions, the requirements securing the greater safety shall apply.

Construction

1. *Classes of construction.* For the purposes of this chapter all buildings or structures shall be classified, with respect to construction, as fireproof, non-fireproof and frame.

2. *Fireproof.* Fireproof buildings or structures are those which are constructed as required in article 17 of this chapter.

3. *Non-fireproof.* Non-fireproof buildings or structures are those which do not conform to the requirements for fireproof buildings or structures, but which are enclosed with walls of approved masonry or reinforced concrete.

4. *Frame.* Frame buildings or structures are those of which the exterior walls or any parts thereof are of wood, or which do not conform to the requirements for fireproof or non-fireproof buildings.

When Buildings are Required to Be Fireproof

1. *New buildings.* Every building hereafter erected shall be a fireproof building, as follows:

(*a*) Every public building over 20 feet high, in which persons are harbored to receive medical, charitable or other care or treatment, or in which persons are held or detained under legal restraint;

(*b*) every other public building over 40 feet in height, or exceeding 5000 square feet in area;

* The Code of Ordinances of the City of New York.

(c) every residence building, except tenements, over 40 feet in height and having more than 15 sleeping rooms;

(d) every tenement house exceeding 6 stories or parts of stories as provided in the Tenement House Law;

(e) every residence building having more than 15 sleeping rooms and exceeding 2500 square feet in area, unless divided by interior partition walls of approved masonry or reinforced concrete into sections of less than 2500 square feet area;

(f) every other residence building over 75 feet in height;

(g) every business building exceeding fifty feet in height, used as a garage, motor vehicle repair shop or oil-selling station within the fire limits or the suburban limits;

(h) every building over 4 stories in height used as a factory as defined in the Labor Law;

(i) every building or structure within the fire limits or the suburban limits used as a grain elevator or a coal pocket;

(j) every business building over 75 feet in height;

(k) every business building within the fire limits or the suburban limits which exceeds an area of 7500 square feet when located on an interior lot or when facing on only one street or 12,000 square feet when facing on 2 streets or 15,000 square feet when facing on 3 or more streets, provided that when any such building is equipped throughout with an approved system of automatic sprinklers, fireproof construction shall be required only when the areas exceed double those herein specified for the respective conditions, and provided also that when any such buildings are divided by approved interior fire walls, fireproof construction shall be required only when any undivided area exceeds 7500 square feet. Buildings of greater areas than herein specified for the respective conditions may, considering location and purpose, be constructed non-fireproof by special permission of the superintendent of buildings, provided they do not exceed 2 stories in height.

2. *Alterations.* (a) By extending. When any building now existing is to be enlarged by extending it on any side so that the enlarged building would exceed the limits of height or area specified in subdivision 1 of this section for a new building, the extension or enlargement shall be constructed fireproof, provided that, in case the existing building is not of fireproof construction, the existing and new portions of the building shall be separated by fire walls.

(b) By raising in height. No building now existing shall be raised in height so as to exceed the limits of height specified in subdivision 1 of this section unless it is fireproof.

When Buildings May Be Non-Fireproof

1. *New buildings.* Except when required by this article to be fireproof, or when permitted by article 5 or article 22 of this chapter to be frame, any building hereafter erected may be non-fireproof.

2. *Alterations.* Except when required by this article to be fireproof, or when permitted by article 5 or article 30 of this chapter to be frame, any building which shall hereafter be enlarged in any manner, may be non-fireproof.

3. *Special fire protection.* In all non-fireproof buildings, hereafter erected or hereafter altered or converted to be used as garages, motor vehicle repair shops or oil-selling stations the columns and girders, unless of fireproof construction, and all wood floor and roof construction shall be covered and protected on all sides with such fire retarding materials and in such manner as may be prescribed by the rules of the Board of Standards and Appeals, except that when such buildings are not more than one story high, without basement or cellar, such protection shall not be required for the roof construction.

Existing non-fireproof buildings heretofore occupied as garages, motor vehicle repair shops or oil-selling stations shall not be required to comply with the provisions of this sub-division, except as may be specifically provided in rules hereafter adopted by the Board of Standards and Appeals.

One-Story Special Buildings

Nothing in this article shall prohibit the use of sheet metal not less than No. 26 gauge in thickness, or other approved incombustible, weatherproof material of such character and thickness as may be prescribed by rules of the Board of Standards and Appeals, for the exterior walls of any building, irrespective of occupancy or location, except when otherwise specifically prescribed by this chapter; provided that such building is not more than one story high above the curb or the surrounding ground level, and that all sides, except for necessary window and door openings, and the roofs of such buildings are of the same material and construction, and provided further that the area does not exceed 1250 square feet, and the side walls 15 feet in height. (Added by ords effective Dec. 26, 1916, May 15, 1917, and July 17, 1917.)

Since types of floor construction are more or less controlled by the class of building in which they are to be built, the natural divisions which result are wood, steel and concrete frame. That is, the main frame will involve one of these three materials as the basis, although the secondary framing may be a combination of materials.

Prob. 97a. If it is desired to erect an apartment house covering an area of 5600□′, six stories high, what class of construction must be used according to the Boston building law? What would be necessary in New York City?

98. Selection of the Type of Floor Panel.

In some kinds of construction, the type of floor panel is more or less settled by common practice, such as for houses of third-class construction, apartments and standard mill buildings of second-class construction. In other types of buildings, however, such as public and office structures, many kinds of floor construction may be used.

When designing floor construction, it should be remembered that simplicity is important, and that a minimum number of sizes should be used. It is better to make several beams which have only slight differences of loading, the same size, thereby en-

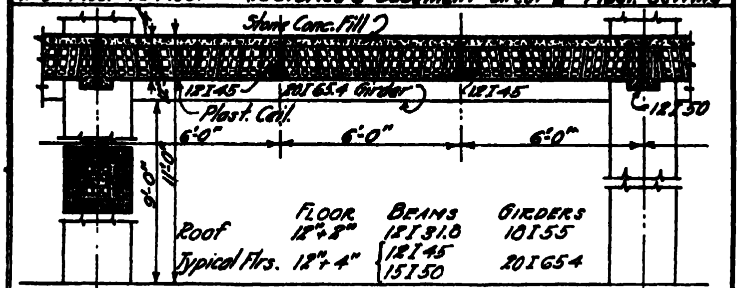
hancing the appearance of the interior and minimizing the number of details.

The selection of a given system for a particular use will depend primarily upon the architectural considerations, the use of the building, conformity to the law, and economic considerations. The locations of the columns and bearing walls are determined by the architectural features of the building. In general, it may be said that they are located where they take up the least valuable space, or where they may be hidden from sight, and so on. The majority of systems use floor beams and girders with a spacing of beams varying from 6′-0″ to 10′-0″. Occasionally cross beams are used only on the column center-lines. These should be used in many cases to stiffen the structure. The direction of the girders is usually that of the short spans so that shallower and more economic construction will result. They should preferably run at right angles to the length of the building.

For certain types of buildings, there is not any

TYPE "A" STEEL & TILE

Panels 18′-0″ x 24′-0″ 11′-0″ Floor to Floor | HOTEL 16 Stories & Basement | 50# L.L. 3″ Tile Partitions on Col. ℄ - Plast. Ceiling

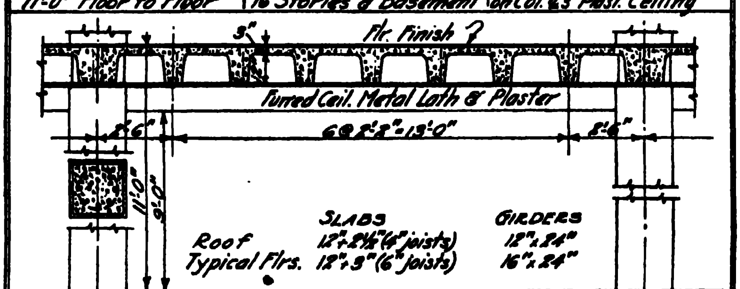


DESCRIPTION	QUANTITY	UNIT COST	COST
Structural Steel			
Columns, Beams & Girders	123,396 lbs.	.05½	6,786.78
Tile Arches & Shoe Tile	7,344 cu.ft.	.50	3,672.00
Column Fireproofing Tile	612 cu.ft.	.50	306.00
Tile Girder Covering	177 cu.ft.	.80	231.80
Concrete Tile 1:3:5 Mix	2,376 cu.ft.	.40	950.40
Cement Finish	7,344 sq.ft.	.07	514.08
Plaster	862 sq.ft.	.80	689.60

80,784 cu.ft. @ $0.163
7,344 sq.ft. @ $1.79
Total Cost = $13,150.06

TYPE "B" CONCRETE JOISTS

Panels 18′-0″ x 24′-0″ 11′-0″ Floor to Floor | HOTEL 16 Stories & Basement | 50# L.L. 3″ Tile Partitions on Col. ℄'s Plast. Ceiling



DESCRIPTION	QUANTITY	UNIT COST	COST
Floor:			
1:2:4 Concrete-Slabs, Joists & Girders	4,896 cu.ft.	.44	2,154.24
Metal Tile	7,344 sq.ft.	.08	587.52
Open Forms	7,344 sq.ft.	.23	1689.12
Closed Forms (Girders)	881 sq.ft.	.35	308.35
Reinforcing Steel	20,525 lbs.	.047	964.68
Columns:			
1:1:2 Concrete	1,338 cu.ft.	.56	749.28
Forms	1,878 sq.ft.	.25	469.50
Reinforcing Steel	9,269 lbs.	.053	491.26
Cement Finish	7,344 sq.ft.	.07	514.08
Metal Lath & Plaster	867 sq.yds.	2.10	1820.70

80,784 cu.ft. @ $0.120
7,344 sq.ft. @ $1.33
Total Cost = $9,740.97

FIG. 156*

* Based upon a paper entitled "Comparison of Types of Fireproof Construction" by Chester L. Post, Consulting Engineer; Vice-President, Condron Co., Chicago, Illinois, — presented to Western Society of Engineers, March 24, 1924.

single system of floor construction which would be used advisedly in all cases, even if the designer were to choose without any ulterior conditions affecting his selection. There are so many variables that there is no definite rule for selecting one type. However, some systems are naturally better adapted than others to the uses, shape, and nature of the structure. In deciding upon the type to use, material assistance is obtained in making the decision by preparing alternate designs for a typical panel and making comparative estimates, based upon the market prices of the respective materials involved, in addition to the cost of erection. Figure 156 shows two comparative estimates. This is not to be taken as a proof of one against the other in this instance, but as an illustration of how to make comparisons. The quantities should be based upon current market prices of materials delivered by truck, and upon average wages for labor, not considering premium wages or unusual rush conditions. The following prices* were used (1924) for making comparative estimates, and **will serve only as a guide for practice problems:**

Structural steel — fabricated (f.o.b. cars)	$90.00/ton
erection	20.00
in place	110.00
Reinforcement (f.o.b. cars)	60.00/ton
(standard increases for sizes below base)	
Standard bending	6.00/ton
Trucking	3.00
Bending stirrups	18.00
Detailed shop drawings	3.00
Spirals for columns	90.00
Placing reinforcement	20.00
Forms — lumber	$47.00/M.B.M.
Removable column forms (incl. erection and removal)	
cylindrical without cap	$15.00/column
with capitals	16.00
Dropped heads	$10.00 each
10″ clay tile	0.25 each
Removable metal domes (rental)	0.08/□′
(no deductions for openings < 100□′)	
Metal lath ceiling plastered direct (complete)	$2.10/□ yd.
Plastering only (2 coat work)	0.90/□ yd.
Concrete (1 : 2 : 4 mix)	0.44/cu. ft.
(1 : 1 : 2 mix)	0.54/cu. ft.

(based upon cement at $2.90 per bbl., less discount and less sacks = $2.45 per bbl., net; crushed stone $2.75 per cu. yd. delivered, sand $2.75 per cu. yd. delivered.

For purposes of comparing quantities, the values are often reduced to a common basis. The cubic feet of concrete, including that in the beams and girders, is divided by the number of square feet in the panel to get an answer of the number of "board feet" of concrete per square foot of panel. Similarly, the forms may be expressed as the number of board feet per square foot of panel, and the reinforcement as so many pounds per square foot. These figures may be increased 25% to allow for the extra filling in columns, beams and girders, and the extra cost of spandrel beams. From the study of various types of construction (mentioned in previous footnotes), the following guides as to economy were derived, mentioned in the order of their economy:

Floors with light loads (such as apartments, hotels, etc.):

(1) metal tile joist
(2) masonry tile joist { terra cotta tile, concrete tile, gypsum tile }
(3) concrete beam and girder
(4) terra cotta arches and steel beams

Light manufacturing:

(1) two-way flat slab
(2) two-way slab and concrete girders } practically the same
(3) one-way slab, beam and girder }

Warehouses:

(1) two-way flat slab
(2) two-way slab and concrete girders
(3) one-way slab, beam and girder

The cost of the construction is not a full measure of economy, however, and **adaptability, as well as maintenance costs, is important.** Some of the other factors which determine the use of a type of floor construction are:

(1) The amount of live load to be carried,
(2) the dead weight of the construction,
(3) the necessity of lateral stiffness,
(4) the ease and speed of erection,
(5) the type of ceiling desired, whether it is to be flush, paneled or suspended,
(6) the fire-resistance of the construction, (one of the most important),
(7) the questions of water tightness and the incident rusting of steel, sound transmission, vermin stopping, and so on,
(8) the coördination with equipment, both in installation and repair, such as
   (*a*) the support of shafting hangers, sprinkler pipes, and the like, on the soffits of the beams, which affects their spacing,
   (*b*) the necessity for carrying pipe and conduit work in the floor fill rather than in between beams,
   (*c*) the bedding of machinery on the surface of the floor, and
(9) the size and arrangement of openings in the floor, and whether pipe sleeves for small openings and inserts may be properly bedded.
(10) The type of foundation, spacing of columns, and clear story heights required will of course affect the cost of the building as a whole.

* From a paper entitled "Analysis of Cost of Types of Fireproof Construction" by Arthur F. Klein, Vice-President, R. C. Wieboldt Co., Chicago, Illinois, — presented to Western Society of Engineers, March 24, 1924.

The system selected should be a modern type of construction. Many kinds of floor framing are now superseded because of the greater advantages and economy of newer ones. The advance in this direction has been rapid in the past century and only a relatively few years ago systems were used which are now practically out of date.*

Prob. 98a. Make a sketch of a rectangular panel 21′-0″ × 16′-0″, showing steel H columns at the four corners. Try different designs for a *total* load of 150#/□′, using the following combinations for the steel beams and girders:

(*a*) girders long way — 4 spaces for beams
(*b*) " " " — 3 " " "
(*c*) " " " — 2 " " "
(*d*) " short " — 3 " " "
(*e*) " " " — 2 " " "

What framing is the most economical? What conclusions do you draw?

99. Tabulation of Loads.

For the purpose of providing an easily checked and complete record of the loads which are used in the design, it is found that an accurate tabulation preceding the calculations for each element of the floor system is of distinct value. These tabulations should be kept in a definite location on the design sheets and should be subdivided into the various items which a designer, mentally, at least, considers in arriving at the loads. For example, the tabulation for the design of a **slab** should be as follows:

L.L. =	60#/□′
Fin. Flr. =	3
Sub. Flr. =	3
2″ Cinder Fill =	16
4″ Concr. Slab =	50 (Assumed)
Pl. Ceiling =	5
T.L. =	137#/□′

In a similar fashion, the tabulation for the load per linear foot on a steel **beam** would be as follows:

Spacing of Beams = 6′-0″

T.L. from floor =	6 × 137	822#/ft.
	Beam + haunch	78
	T.L. =	900#/ft.

A very common error in beam load tabulations is to add the weight of the beam and haunch into the square foot allowance. This is obviously incorrect. Another common error which creeps into floor load calculations is the development of the load per linear foot from the load per square foot in joist spans. For example, a steel tile, concrete joist, ribbed slab in which the joists are 25″ o.c., would give a load per linear foot of $\frac{25}{12}$ times the load per square foot.

In connection with the tabulation of loads for **girders,** great care should be taken to make sure that the reactions of the beams framing into the girder are considered. Thus, if two beams, framing on each side of the girder and which have the same span and load for example, have their reactions transferred to the girder, the concentration will not be the reaction of one beam but that of both beams, or, in other words, the total load on one beam. Every girder, especially in fire resisting construction, has a definite and appreciable weight which should not be ignored in the calculations.

All of these suggestions of course are apart from those special conditions imposed by partitions, floor openings, hangers, and such other instances of loading which are not always a part of the regular floor system design. In the discussion of the types of floor systems in the following chapters, it will be interesting to note the methods of tabulation.

Without efficient tabulation, it is almost impossible to interpret the usual building code as affecting column design. Here are found many variations of suggested live load reductions, each in its own way imposing a definite system of tabulation.†

Prob. 99a. Tabulate the loads for the floor, typical beam and girder for the following data:

Live load 150#/□′.
1″ granolithic finish floor, 5″ slab, ceiling plastered direct. Spacing of beams 7′-0″, size 12 I 31.8 fireproofed. Panel 21′-0″ square, girders 20 I 65.4, fireproofed.

Make a sketch for the loading on the girder.

* In 1855, one type of fire resisting construction consisted of iron joists composed of two curved sheets and two flat strips, riveted top and bottom, resting upon iron girders. Above the joists, corrugated iron sheets supported a concrete fill on which the finish flooring was laid. Such a system would be obsolete in modern practice.

† See Chapter 20.

CHAPTER 7

REINFORCED CONCRETE SLABS

100. Comments.

It is not the intention to discuss the complete theory of reinforced concrete beams and slabs here, as this may be found in a number of treatises on this subject. However, an engineer designing steel-framed buildings often employs a type of floor or roof construction which involves the use of reinforced concrete slabs. He should be able to design the slabs as a part of his work: — to define the slab thickness and how this concrete is to be reinforced (the sizes of rods or mesh, and how the rods are to be bent and placed). Such design involves only the simpler elements of the concrete theory.

101. General Theory.

The medium which carries the finish flooring and transmits the loads to the beams in some forms of construction is called the slab. A slab is designed in the same manner as a concrete rectangular beam except that the width, b, is generally considered to be that of a 12″ strip. A strip of this width is taken principally as a matter of convenience because the load per square foot, w_0, is then the load per running foot, w, for the slab design. If any one foot width is strong enough to span the opening, the slab as a whole is safe. The required depth may be found by applying

$$M = K \cdot b \cdot d^2, \text{ in which}$$

K = a constant, depending upon the allowable stresses in the concrete and steel. Values are given in Table 34.

d = the distance in inches from the top of the slab to the center of the reinforcement, and

b = 12″, the width of the strip of slab, and

M = the maximum bending moment for the strip, expressed in inch-lbs.

This formula may be more conveniently expressed as

$$\frac{\overline{M}}{K \cdot b}$$

The area of steel required for a one-foot strip may then be calculated by using

$$A_s = \frac{M}{f_s \cdot j \cdot d}, \text{ in which}$$

A_s = the area in sq. ins. of reinforcement required per foot of width of the slab,

f_s = the maximum allowable tensile stress in the reinforcement, in #/□″, and

j = a constant, which for all practical purposes may be used here as $\frac{7}{8}$.

The shear is generally relatively small, so that the intensity of shear is usually below the allowable for the concrete and no web reinforcement is required. Since this is true in the majority of cases, the **shear investigation** is generally **omitted** in the design of slabs. As the typical slab is reinforced with a relatively large number of small rods and the shear is small, the **bond stress** is usually within safe limits and this **calculation** is also **omitted.** Consequently, the design of slabs is a relatively simple procedure.

When the **finished floor** is to be **granolithic,** that is, of a concrete surface, it may be made in two ways. One is to float the finish **integral** with the slab and the other is to apply a finish coat later. The latter method is more commonly used as it is more convenient and a better result can be obtained.† Since such a **finish is applied later,** it should **not** be **counted upon as** a part of the **effective depth** of the slab, but as merely additional load. If it is positively known that the finish is to be integral the depth may be taken as effective to the top of the finish floor. Usually a large majority of the structural design work is completed before the specifications are finally prepared so that the safer practice is to assume that the finish will be applied later.

102. Allowable Compressive and Tensile Stresses.

The maximum allowable fibre stress in compression for flexure is based upon the ultimate compressive strength of the concrete. In order to refer to compressive strengths for uniform conditions, the data should be based on the 28-day compressive

* Some engineers prefer to use the formula $d =$ ' as a more convenient application: This is the same solution as $d = \sqrt{\frac{M \text{ (ft.-lbs.)} \times 12}{K \times 12}}$.

† There are advantages with either method. A finish cast later requires careful workmanship to obtain a good bond. However, a better finish is obtainable, and it is more convenent in casting integral bases, thresholds, and so on.

strengths of 6″ϕ × 12″ cylinders, made and stored under standard laboratory conditions,* using the same consistency and mix as in the field. There has been considerable tendency in the past few years to specify concrete as 1500#, 2000#, 2500#, 3000#, etc., for example, and to vary the proportions of the cement, fine aggregate, and coarse aggregate to obtain such strengths.†

Another method which is still used in the absence of definite knowledge in advance of the construction as to just what strength may be expected, or what materials are to be used, is that of establishing maximum compressive strengths according to the arbitrary mixes of concrete which are specified. The following table represents average practice.

The arbitrary method of using average values is misleading, as the strength of the concrete is dependent upon many factors such as:

(1) the quality of the cement,
(2) the proportion of the cement per unit volume,
(3) the character and size of the aggregate,
(4) its density,
(5) the relative amounts of mixing,
(6) its age, and
(7) the nature of the seasoning.

However, the above values are the most commonly used because of the reasons given below.

TABLE 33

COMPRESSIVE STRENGTHS OF DIFFERENT MIXTURES OF CONCRETE

(In Pounds per Square Inch)

Aggregate	1 : 3‡	1 : 4½‡	1 : 6‡	1 : 7½‡	1 : 9‡
Granite, trap rock	3300	2800	2200	1800	1400
Gravel, hard limestone and hard sandstone	3000	2500	2000	1600	1300
Soft limestone and sandstone	2200	1800	1500	1200	1000
Cinders	800	700	600	500	400

‡ These values are based upon the arbitrary mixes, 1-1-2, 1-1½-3, 1-2-4, 1-2½-5 and 1-3-6 respectively but the combined volume of fine and coarse aggregates measured separately should not exceed the figures given.

There is a tendency in recent practice toward more careful analysis of the aggregate and the establishment of more accurate proportions than those usually represented by the arbitrary mixes. The ideal way of establishing the ultimate compressive strength of the concrete is by averaging the results of tests of a series of standard test cylinders. These specimens are made of the materials which are to be used, mixed in the proper proportions and consistency suggested by careful laboratory study. However, the design of a structure is generally completed except for details before the contract for construction is awarded. The nature of the aggregates to be used will depend upon the sand bank and gravel pit or crusher from which the contractor intends to take his supply. The problem then becomes one of making the concrete actually cast as near in strength to that specified as possible, if the method of fixing an ultimate strength is used, but the human equation also enters even though the natural qualities of the aggregate are carefully considered. Therefore many building codes specify the ultimate strengths according to arbitrary mixes, similar to the table above.

The **working compressive stress** is quite universally expressed as some proportion of the ultimate strength of the concrete at the age of 28 days. The following excerpts illustrate this.

SPECIFICATION CLAUSES (J. C.)

Flexure	Extreme fiber stress in flexure	$0.40 f_c'$
	Extreme fiber stress adjacent to supports in continuous beams	$0.45 f_c'$
Boston‡ Building Law	Compression on extreme fiber in bending shall not exceed thirty-two and five-tenths per cent of the compressive strength fixed by this act: provided, however, that adjacent to the supports of continuous beams or slabs thirty-seven and five-tenths per cent may be used.	

The strength of concrete increases with age, so that the factor of safety is larger in a concrete structure after a period of time than it was when the structure was first built. Many building codes make allowance for this characteristic, if it is desired to use a member for an additional load later.

SPECIFICATION CLAUSE‡

Concrete 1 Year Old Concrete one year old shall be considered to have a compressive strength twenty-five per cent greater than that given in the table for concrete of the same grade and proportions.

Illustrative Prob. 102a. What is the maximum allowable compressive stress in flexure for a 2000# concrete,§ J. C. Rules?

$$f_c' = 2000\#/\square'' \qquad f_c = 0.40 \times 2000 = 500\#/\square''.$$

Illustrative Prob. 102b. What is the maximum allowable compressive stress in flexure for a 1 : 6 concrete, Boston Law? That adjacent to the supports of continuous beams?

$$f_c' = 2200\#/\square' \qquad f_c = 0.325 \times 2200 = 715\#/\square''$$

$$\text{Adjacent to supports } f_c = 0.375 \times 2200 = 825\#/\square''.$$

* "Tentative Methods of Making Compression Tests of Concrete" (Serial Designation: C39-21T) of the A.S.T.M. (given in Appendix XIII of the J. C. Report also), "Standard Methods of Making and Storing Specimens of Concrete in the Field" (Serial Designation: C31-21) of the A.S.T.M.

† J. C. Report, 1921. Table 4 gives the proportions in which the materials should be mixed. It is also stated that the purpose is twofold:

1. To furnish a guide in the selection of mixtures to be used in preliminary investigations of the strength of concrete from given materials.
2. To indicate proportions which may be expected to produce concrete of a given strength under average conditions where control tests are not made.

‡ The Building Law of the City of Boston. (The ultimate strength of concrete increases about one-third at the age of 6 months — see Bulletin #107, Univ. of Wisconsin.)

§ This expression is commonly used to designate a concrete which has an ultimate compressive strength of such a value in #/□″.

The maximum **allowable tensile stresses** for steel reinforcement such as bars,* wire in various forms,† and small structural steel shapes‡ are well established because of the reliable quality of manufacture.

SPECIFICATION CLAUSES (J. C.)

Tension in Steel (206)

(a) Billet-steel bars:

1. Structural steel grade	16,000 lb. per sq. in.
2. Intermediate grade	18,000 lb. per sq. in.
3. Hard grade	18,000 lb. per sq. in.
(b) Rail-steel bars	16,000 lb. per sq. in.
(c) Structural steel	16,000 lb. per sq. in.
(d) Cold-drawn steel wire	18,000 lb. per sq. in.

These working stresses are based on a factor of safety of 4. Thus for structural steel grade bars,

Ultimate Strength = 65,000#/□″ (Average).

$$f_s = \frac{65,000}{4} \quad 16,000\#/\square'' \text{ (Allowable).}$$

The Joint Committee recommends the use of intermediate grade billet-steel bars, although many engineers prefer to use structural steel bars as many building codes limit the allowable tensile stress for bars to 16,000#/□″ maximum, and structural grade steel is naturally cheaper than intermediate grade steel, if such limitations are given. The former grade has greater elastic qualities than the latter. Although hard grade steel has a higher ultimate strength than the other steels, it has a tendency toward brittleness, so that the working stress is limited to 18,000#/□″. The authors believe that all such stresses should be limited to 16,000#/□″, except cold-drawn steel wire, as the moduli of elasticity are practically the same, and the higher carbon grades are not always as reliable in all respects, particularly rail steel. Cold-drawn steel wire has a high ultimate strength and is comparatively elastic, so that the allowable stress is also given as 18,000#/□″. Some specifications allow a maximum of 20,000#/□″ instead; others allow increased values for steel used as reinforcement in slabs.

SPECIFICATION CLAUSE §

Tension in Reinforcing Steel

The tensile or compressive stress in steel shall not exceed sixteen thousand pounds per square inch in rods and twenty thousand pounds per square inch in drawn wire and other approved cold stretched fabric, except that in slabs of stone concrete the tensile stress in rods shall not exceed eighteen thousand pounds per square inch, and in drawn wire or other approved cold stretched fabric it shall not exceed twenty-two thousand five hundred pounds per square inch.

* "Standard Specifications for Billet-Steel Concrete Reinforcement Bars" (Serial Designation: A15-14), A.S.T.M., or "Standard Specifications for Rail-Steel Concrete Reinforcement Bars" (Serial Designation: A16-14), A.S.T.M.

† "Tentative Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement" (Serial Designation: A82-21T), A.S.T.M.

‡ "Standard Specifications for Structural Steel for Buildings" (Serial Designation: A9-21), A.S.T.M.

§ The Building Law of the City of Boston.

A feature of the discussion of the working stresses in reinforced concrete is the relative **factor of safety.** If this were based upon the strength of the concrete at the age of 28 days, there would not be a sufficient amount of steel to develop the strength of the concrete when it was six months old. Thus a factor of safety of 3.5 for concrete 28 days old **becomes** one of **4.5 to 5 for concrete six months old.** The strength of the combination is dependent upon the yield point of the steel and not its ultimate strength, for once the steel yields materially, the concrete tends to disintegrate. The yield point of steel is from 30,000 to 35,000#/□″, and assuming that the steel could elongate somewhat before the crushing of the concrete became serious, the factor of safety referred to the yield point is nearer 2½ or 3, instead of 2 (32,000 average ÷ 16,000 = 2). Hence the **resulting factor of safety** of the combination **averages about four,** which is consistent with the general design.

Prob. 102c. What is the maximum allowable compressive stress in flexure for a 1 : 1½ : 3 concrete, gravel aggregate, J. C. Rules?

103. Design Factors.

In any given case, it is necessary to establish the value of the design factor, K, according to the working stresses for the concrete and steel to be used. Table 34 gives the formulas used and the values resulting for several combinations of stresses. For the common mixes of stone concrete, n, the ratio of the modulus of elasticity of the reinforcement to that of the concrete, is usually 15. For cinder concrete, n = 40 usually.

Illustrative Prob. 103a. What is the value of K for 2500# concrete, intermediate grade reinforcement, and J. C. Rules?

$$f_c' = 2500\#/\square'' \qquad f_c = 0.40 \times 2500 = 1000\#/\square''$$
$$f_s = 18,000\#/\square'' \qquad n = 12$$
$$p \quad \frac{0.50}{\frac{18,000}{1000}\left(\frac{18,000}{12 \times 1000} + 1\right)} \quad 0.0111$$
$$k = \sqrt{2\,(0.0111)\,(12) + [0.0111\,(12)]^2} - 0.0111\,(12)$$
$$k = 0.401$$
$$j = 1 - \frac{0.401}{3} = 0.866$$
$$K = \tfrac{1}{2} \cdot f_c \cdot k \cdot j = \tfrac{1}{2}\,(1000)\,(0.401)\,(0.866) = 173.2$$
$$K = p \cdot f_s \cdot j = 0.0111\,(18,000)\,(0.866) = 173.2$$

Prob. 103b. Calculate the value of K for 2200# concrete, structural grade reinforcement and J. C. Rules.

104. Beam Reinforcement.

The principal forms of reinforcement for concrete are **round rods** and **square bars** of steel. These are of two types, **plain** and **deformed.** The deformed bar is one on which lugs or indentations are rolled

TABLE 34

DATA FOR CONCRETE BEAM DESIGN

$$p = \frac{0.5}{\frac{f_s}{f_c}\left(\frac{f_s}{n.f_c}+1\right)} \qquad k = \sqrt{2\,p.n+(p.n)^2}-p.n$$

$$j = 1 - \frac{k}{3}$$

$$K = \tfrac{1}{2} f_c.k.j = p.f_s.j \qquad M = K.b.d^2$$

Working Stresses		Ratio of Moduli $n = 15$				Remarks
f_s	f_c	p	k	j	K	Typical Code Requirements
16,000	500	.0050	0.319	0.894	71.3	
	550	.0058	0.339	0.887	82.3	
	600	.0067	0.358	0.881	94.4	
	650	.0077	0.378	0.874	107.4	Old J. C. — comp. @ mid-span
	700	.0087	0.397	0.868	120.6	
	715	.0089	0.403	0.865	124.0	Boston Law — comp. @ mid-span
	750	.0097	0.414	0 862	133.8	Old J. C. — comp. @ supports
	800	.0107	0.429	0 857	146.7	New J. C. — comp. @ mid-span
	825	.0112	0.436	0.855	154.0	Boston Law — comp. @ supports
	880	.0126	0.451	0 854	170.0	New J. C. — 2200# concrete
	900	.0128	0.457	0 844	173.0	New J. C. — comp. @ supports
	600	.0056	0.334	0.889	89.6	
	650	.0063	0.351	0.883	100.0	
	700	.0072	0.368	0.877	114.0	
18,000	715	.0074	0.373	0 876	117.0	Boston Law — slabs — rod reinf.
	800	.0088	0.398	0.867	137.0	New J. C. — slabs — mesh reinf.
	825	.0093	0.410	0.836	144.0	Boston Law — slabs — comp. @ supports
20,000	500	.0034	0.272	0 909	61.8	
	550	.0040	0 292	0 903	72.2	
	600	.0047	0.311	0 897	83.7	
	650	.0053	0.328	0 891	94.4	
	700	.0060	0.344	0.885	106.2	
	715	.0062	0.347	0 882	110.1	Boston Law— slabs — mesh reinf.
	750	0067	0 359	0.880	117.9	
	800	.0075	0.374	0.875	130.9	
18,000	240	.0126	0.231	0.923	25*	New J. C. — slabs — mesh reinf.

in one manner or another. There are many types of deformed bars on the market, among which are the **American,** manufactured by the American System of Reinforcing; **Corrugated,** manufactured by the Corrugated Bar Company; **Havemeyer,** manufactured by the Concrete Steel Company; **Rib,** made by the Truscon Steel Company; and the **Inland,** made by the Inland Steel Company. Figure 157 shows some of the common kinds. The protection against plain rods ultimately slipping under stress depends upon their adhesion to the concrete. This resistance is called **bond.** Deformed bars have an additional† protection furnished by **mechanical** **bond.** Previously, square-twisted bars were employed, but their use has been largely superseded by deformed bars.

The steel from which rods are manufactured may be made by the Open Hearth or Bessemer process (Arts. 1 and 2). It is generally conceded that the Open Hearth process is the better. The material may be **Billet Steel‡** or **Rail Steel.§** Many engi-

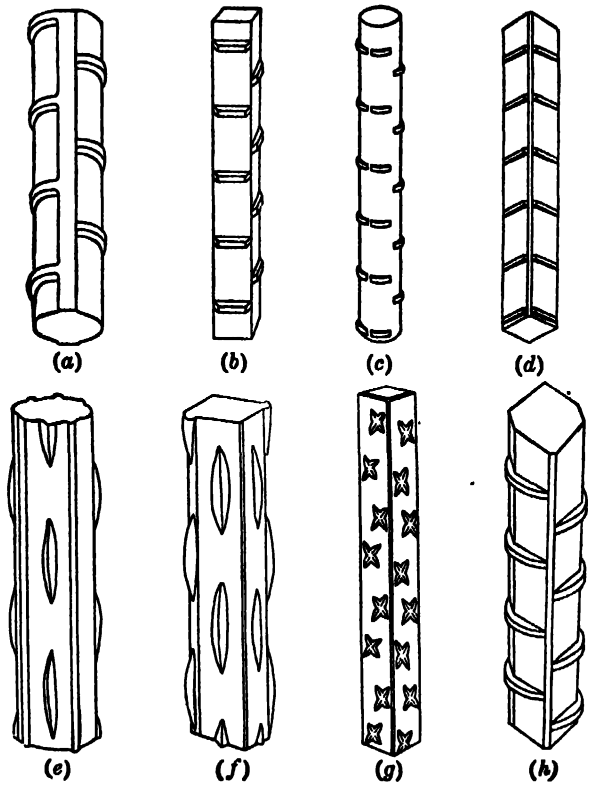


FIG. 157. REINFORCING RODS AND BARS

(a) Corrugated bar
(b) Corrugated bar
(c) American bar
(d) American bar
(e) Havemeyer bar
(f) Havemeyer bar
(g) Inland bar
(h) Rib bar

neers prefer to specify billet steel. The rods are made in different grades, namely, **structural, medium** and **high-carbon.** The medium steel is considered by many to be the best grade to use in the majority of cases. High-carbon steel, although stronger in tension, is apt to be brittle. This is often an undesirable feature but in heavy members the concrete itself will absorb a large portion of the shocks and the vibration, so that the steel is somewhat protected in such cases. More careful inspection of such rods is required as they may be damaged before the concrete is cast, so that in general little economy is

* 1 : 2 : 4 Cinder Concrete, $n = 40$.

† This additional protection is principally in the resistance of the bar to ultimate slipping. The stresses which will cause initial slip for the plain and deformed rods are not as widely separated.

‡ "Standard Specifications for Billet-Steel Concrete Reinforcement Bars" (Serial Designation: A15-14) of the A.S.T.M.

§ "Standard Specifications for Rail-Steel Concrete Reinforcement Bars" (Serial Designation: A16-14) of the A.S.T.M.

gained by their use. The quality* of steel as manufactured is very uniform so that tests are not frequently required to determine whether or not it is satisfactory in elastic properties.

The **common sizes** of reinforcing rods vary from 1/4 to 1 1/4" in multiples of 1/8". One-sixteenth sizes are made but they are difficult to obtain because of infrequent rollings. If an engineer calls for the latter sizes, substitutions will often be made in order to use available stock and to expedite the progress of the job. Table 35 gives the common sizes, areas and weights of rods universally adopted† at present. It should be noted that some deformed bars are heavier than others on account of the lugs. The cross sectional areas, however, are all practically the same because the patented bars are planned to have areas equivalent to plain, round or square bars of the same size.‡

TABLE 35

AREAS, PERIMETERS, AND WEIGHTS OF RODS

	Rounds			Squares		
	Area	Perimeter	Weight	Area	Perimeter	Weight
3/8	0.11□"	1.18"	0.38#/ft.	.....	....	.....
1/2	0.20	1.57	0.67	0.25□"	2.00	0.86#/ft.
5/8	0.31	1.96	1.05	....	....	.....
3/4	0.44	2.36	1.52	.....	....	.....
7/8	0.60	2.75	2.06	....	....	.....
1	0.78	3.14	2.69	1.00	4.00	3.43
1 1/8	...	....	....	1.27	4.50	4.34
1 1/4	.....	...	....	1.56	5.00	5.31

The matter of the selection of rods for reinforcement is a problem of economy. The number of sizes and lengths used should be kept as low as possible. Small sizes are used for slabs, larger sizes for heavy beams and occasionally the largest sizes have to be used for heavy columns. The medium sizes should be employed as much as possible because **base prices and extras** influence the cost. Certain sizes and lengths are classed for a base price and the others are priced by extras according to size. Table 36 shows the typical relations of such prices.

In some cases, engineers give only the areas of steel required for the various parts of members. The sizes and spacing of rods to provide the required area in such cases must be furnished by the reinforcing steel contractor. This is done to save the time of the engineer and to allow the contractor to supply the steel according to his local stock, if possible. This procedure is not considered the best practice and the time spent by the engineer in showing the rod sizes, spacing, and typical bend points will usually prove advantageous.

TABLE 36

EXTRAS ON REINFORCEMENT

Size.	(Per 100#)
3/4 and larger	Base
5/8 to 11/16	$0.05
1/2 to 9/16	.10
7/16	.20
3/8	.25
5/16	.35
1/4	.50
Length.	
Over 48 inches and less than 60 inches	$0.05
24 inches to 48 inches inclusive	.10
12 inches to 23 inches inclusive	.20
Under 12 inches (on application)....not less than	.30
Tonnage.	
Over 1000 pounds but less than 2000 pounds	$0.15
Less than 1000 pounds	.35

Illustrative Prob. 104a. If a beam were reinforced with 2-7/8" φ rods bent, and 1-7/8" φ rod straight, what number and size of square bars could be used in place of the round?

$$2\text{-}\tfrac{7}{8}''\ \phi \text{ area} = 2 \times 0.60 = 1.20$$
$$1\text{-}\tfrac{7}{8}''\ \phi \text{ area} = 1 \times 0.60 = \underline{0.60}$$
$$1.80$$

Substituting

$$2\text{-}\tfrac{3}{4}''\square \text{ bars} = 1.12$$
$$1\text{-}\tfrac{7}{8}''\square \text{ bar} = \underline{0.76}$$
$$1.88\square''$$

Prob. 104b. If 7/8" φ rods were not available and you had 5/8" φ and 3/4" φ, how many of each would you use to give the same cross sectional area as in Illustrative Prob. 104*a*?

Prob. 104c. Arranging the steel in a double layer with 2" between layers and bars 2" o.c., substitute rods of 1" φ or under for 2-1 1/8" φ straight and 2-1 1/8" φ bent, keeping the same percentage of bent steel.

Prob. 104d. If you had 5/8" φ rods, 4" o.c. specified for a slab, how many rods would there be per foot of width? What cross sectional area of steel per foot is obtained? If you had to use 1/2"□ bars instead, how many inches on center should they be?

105. Simple Slabs.

Slabs which span only a **single opening** and also are simply supported are rather an exception in an average building. They may, however, occur in special cases such as when there are openings in a floor either side of a pair of parallel beams, or in bridge floors, platforms, coal pockets, and so on. Usually there is some restraint at the supports of

* "Standard Specifications for Billet-Steel Concrete Reinforcement Bars" (Serial Designation: A15–14) of the A.S.T.M.

† The Joint Conference of Representatives of Manufacturers, Distributors, and Users of Concrete Reinforcement Bars held in the Department of Commerce, Washington, D. C., September 9, 1924, unanimously adopted the areas and sizes of reinforcement bars in Table 35, with the addition of the 1/4 in. round bar, area, 0.049 sq. in.; to become effective as applying to new production, January 1, 1925; every effort to be made to clear current orders and existing stocks of eliminated areas before March 1, 1925.

‡ Some deformed bars are defined by this method, such as #125 Ovoid (1 1/4" φ equivalent).

such slabs so that the bending moment may be calculated by $M = \frac{w \cdot L^2}{10}$ or $M = \frac{w \cdot L^2}{12}$, depending upon local conditions. The typical slab generally extends over two or more spans so that the moment coefficients for continuous beams are used in one form or another.

In certain types of construction the spans of the slab may range from 4′-0″ to 6′-0″, only. In these short spans the slabs become rigidly supported if they are well bonded to the beams, as should be the case in monolithic construction. The adjoining slabs give a great stiffening effect to the slab under consideration even in the case when the slabs are supported by steel beams. For such conditions, **reinforcement against negative moment** is **hardly needed,** and straight rods in the bottom of the slab, as in the case of simply supported slabs, are considered sufficient. Such rods should be lapped and the laps should break joints to prevent local cracks. The length of the laps should be sufficient to develop bond. When such slabs are built fully continuous and on short spans they may be designed for a moment of $\frac{w \cdot L^2}{12}$ and the steel proportioned as described above.

When the spans exceed 6′-0″, the stiffening action of adjoining slabs is less effective, and **provision for negative moment** must be made by the use of reinforcement in the top of the slab over the supports. This is generally accomplished by bending up part of the rods provided for positive moment to the top of the slab and extending them into the adjacent span. The inclination of the **bent rods** for slabs is quite universally **made 30° with** the **horizontal.** If the rods were bent at too small an angle they would run too great a distance before reaching the top of the slab. On the other hand, if the rods are bent at too sharp an angle there is a tendency of the slab to crack. The points at which to bend the rods up, theoretically, should be at the points of inflection of the moment curve. This degree of accuracy is hardly necessary for ordinary slab design and the usual practice is to make the **bends at** the **quarter-points** of the center to center **span.*** The bent **rods** are usually **extended to** the **quarter-points of** the **adjacent span.** In this way sufficient bond resistance is developed, and the rods bent up from either side of a support may be counted upon as resisting the negative moment. The top of the slab should theoretically be reinforced transversely to provide for the negative bending, at right angles to the direction of the main slab reinforcement and to help stiffen the supporting beam. The steel in the top of the beam itself is generally considered sufficient to provide such stiffening action.

* Some designers prefer to use the fifth-point of the clear span.

Illustrative Prob. 105a. Determine the theoretical depth of slab required to span 7′-0″ and carry a live load of 250#/□′. Slab is fully continuous and is to have a 1″ granolithic finish. Maximum allowable f_c = 650#/□″ and f_s = 16,000#/□″.

Since concrete slabs have such a relatively large dead weight, such weight has to be included in the load per square foot. Since the required thickness of slab is unknown at the outset of the design, a weight has to be assumed,

L.L.	= 250#/□′		$K = 107.4$
1″ Grano. Fin.	= 12		
5″ Slab	= 62 (assumed)		
T.L.	= 324#/□′		$M = \frac{w \cdot L^2}{12}$ ft. lbs.

$$M = 1.0\, w \cdot L^2 = 1.0 \times 324 \times (7)^2 = 15{,}900''\#$$

$$d = \sqrt{\frac{M}{K \cdot b}} = \sqrt{\frac{15{,}900}{107.4 \times 12}} = 3.51''.$$

When the theoretical depth of slab has been calculated, the **actual depth** is generally taken **to** the **nearest multiple of** ½″ above that required, unless the theoretical value is very near a ½″ value. An obvious reason for this is that the character of the construction does not call for fine degrees of measurement. The increased depth will also serve to lighten the reinforcement required. In the above case, d (actual) may be taken as 3½″ but if d (theoretical) had calculated, say, 3.72″, then the value should be taken as 4″.†

The reinforcement must be given proper fire protection. It is customary to **provide 1″ below the center-line of steel.** The overall depth or thickness, t, in the above problem should then be $3\frac{1}{2} + 1 = 4\frac{1}{2}''$. The minimum thickness of slab allowable is often limited, to avoid extremely thin members.

SPECIFICATION CLAUSE‡

Minimum Thickness In roof slabs the total depth shall not be less than three inches and in floor slabs four inches.

When the thickness of slab has been calculated the actual **weight** should be **checked against** the originally **assumed weight** of the slab. These two values should check within 10%. Otherwise the slab should be redesigned.

Illustrative Prob. 105b. Calculate the required area of steel for the Illustrative Prob. 105*a*.

$$A_s = \frac{M}{f_s \cdot j \cdot d} \qquad d = 3.5''.$$

$$A_s = \frac{15{,}900}{16{,}000 \times \frac{7}{8} \times 3.5} = 0.324\square''.$$

The above value means that 0.324□″ of steel reinforcement must be supplied for every 12″ width of slab.§ The sizes of **rods** most **commonly** em-

† In small buildings of less than 50,000 square feet in total floor area, it will probably be as simple and as economical to make slabs the nearest 1″ above the theoretical value in thickness. For larger construction, the ½″ variation is advised for economy. Recently, ½″ variations were discarded in favor of ¼″ increments, and it is quite possible that 1″ variations will be generally used at some future date.

‡ Building Law of the City of Boston.

§ This is the width which was used in the determination of the required depth of slab.

ployed for slabs are $\frac{3}{8}''\ \phi$, $\frac{1}{2}''\ \phi$, $\frac{1}{2}''\square$ and $\frac{5}{8}''\ \phi$. Sizes larger than these are only used for exceptionally heavy slabs. The most commonly used are the $\frac{1}{2}''\ \phi$ and $\frac{5}{8}''\ \phi$. When the value of A_s is known, it is good practice to solve for the spacings corresponding to one or two sizes of rods so that the most satisfactory and economical arrangement may be made.* The rods should not be placed too closely together, as the aggregate must be allowed to pass between them. In practice the rods are seldom placed at the minimum of three diameters on centers and 4″ o.c. is usually about the **minimum spacing.** On the other hand, the slab rods should not be spaced too far apart as the portion of concrete between the rods may tend to act as an individual beam and thus be subjecting the concrete to excessive tensile stresses.

SPECIFICATION CLAUSE†

Maximum Spacing Slab reinforcement bars in tension shall be not farther apart horizontally than two and one-half times the total thickness of the slab.

The spacing of rods is generally kept less than such a limit, however, in order to keep the rod sizes small and to keep the rods more numerous.

Illustrative Prob. 105c. Select an arrangement of rods for the area of steel required in Illustrative Probs. 105*a* and 105*b*.

A_s required $= 0.324\square''$ $\qquad \frac{3}{8}''\ \phi = 0.11\square''$ (Table 35).

$$\frac{0.324}{0.11} = 3 \text{ rods in every } 12''$$

$$\frac{12}{3} = 4'' \text{ o.c.}$$

This calculation may be made in one step if desired. Thus

$$\text{Spacing} = \frac{12'' \times \text{area of one rod}}{A_s} = \frac{12 \times 0.11}{0.324} = 4.06'' \text{ o.c.}$$

As suggested, two or three rod sizes will be tried, and the best combination selected.

$\frac{3}{8}''\ \phi$ — 4.1″ o.c. or 4″ o.c.
$\frac{1}{2}''\ \phi$ — 7.1″ o.c. or 7″ o.c. ← Use
$\frac{5}{8}''\ \phi$ — 11.3″ o.c. or 11″ o.c.

The actual figure to use for the **spacing** of rods should be the **nearest $\frac{1}{2}''$ below that theoretically required,** unless the theoretical figure is reasonably close to a $\frac{1}{2}''$ multiple. This spacing is sometimes varied after 6″ to the nearest 1″. Too fine a degree of measurement would be unreasonable in locating steel and it would add considerably to the cost to insist upon exact location. Steel, however carefully located, will probably not remain in its exact position due to the possible walking on the steel before the concrete is cast and because of the tamping of the concrete into place. Figure 158 will be found useful in determining arrangements of rods to supply a given area of steel.

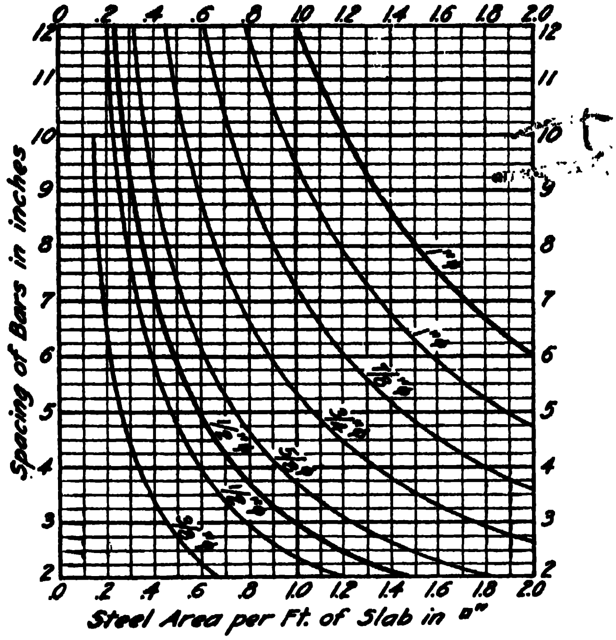


FIG. 158

"When areas of concrete too large to expand and contract freely as a whole are exposed to atmospheric conditions the changes of form due to shrinkage and to action of temperature are such that cracks may occur in the mass unless precautions are taken to distribute the stresses so as to prevent the cracks altogether or to render them very small. The distance apart of the cracks, and consequently their size, will be directly proportional to the diameter of the reinforcement and to the tensile strength of the concrete, and inversely proportional to the percentage of reinforcement and also to its bond resistance per unit of surface area. To be most effective, therefore, reinforcement (in amount generally not less than one-third of one per cent of the gross area) of a form which will develop a high bond resistance should be placed near the exposed surface and be well distributed. Where openings occur the area of the cross section of the reinforcement should not be reduced. The allowable size and spacing of cracks depend on various considerations, such as the necessity for water-tightness, the importance of the appearance of the surface, and the atmospheric changes."‡

The above paragraph refers in general to concrete exposed to atmospheric conditions. The average slab is not usually subjected to great temperature changes or to conditions of excess moisture. However a certain amount of steel is desirable to prevent local cracks due to shrinkage. The reinforcement

* For information relative to rod sizes, base prices, and so on, see Art. 104.

† The Building Law of the City of Boston.

‡ Excerpted from the J. C. Report of 1916.

quired for tension supplies a large amount of this. Nevertheless a slab should be **reinforced longitudinally** to prevent cracks in the concrete. Longitudinal reinforcement is more important in long spans than in short ones. The third of one per cent referred to in the preceding paragraph applies to concrete which is exposed to the weather. This is an empirical value but it is based upon sound judgment and experience. Since the average slab is not exposed to great temperature changes, and there is already reinforcement in one direction, a lesser amount of longitudinal steel will be effective. It is quite **common practice** to place **$\frac{3}{8}''$ ϕ rods** from **18″ to 24″ o.c.** in a longitudinal direction, depending upon the size and importance of the slab. Some engineers consider it to be more economical to use $\frac{1}{2}''$ ϕ rods 24″ o.c. in all cases, for the reason that the base price on these rods offsets the difference in weight between them and the $\frac{3}{8}''$ ϕ rods, and that the practice results in one less rod size to be considered in many cases. The strength of the longitudinal steel in a " one-way " reinforced slab is not counted upon in the design. The longitudinal rods also serve the perhaps more important purpose of acting as **spacers** for the main bars. The reinforcement in the two directions should be fastened together at frequent intervals to keep the bars at the proper spacing and to aid in tying the slabs together as a whole.

The vertical shear at the supports of the slab in Illustrative Prob. 105a, in the direction of the transverse reinforcement, is 324#/ft. × 3.0* ft. = 972#. The intensity of vertical shear is then

$$v = \frac{V}{b \cdot j \cdot d} = \frac{972}{12 \times \frac{7}{8} \times 3.5} = 26.4\#/\square'' \text{ O.K.}$$

Thus the shearing stress is well within the allowable for the concrete by itself and no web reinforcement is necessary. The rods bent up for negative moment will of course provide some protection against diagonal tension. The shear is usually within the allowable and **slabs** do **not** have to be **reinforced for shear** unless they are exceptionally heavy. The bond stress on the transverse steel for the above data (neglecting the longitudinal steel) may be calculated in the following manner:

$\frac{1}{2}''$ ϕ rods — 7″ o.c. Consider a 7″ strip of slab then

$$V = 972 \times \frac{7}{12} = 566\#. \quad \text{One } \tfrac{1}{2}''\phi \text{ rod is effective in this width.}$$

$$u = \frac{V}{j \cdot d \cdot \Sigma_0} = \frac{566}{\frac{7}{8} \times 3.5 \times 1.57} = 118\#/\square''. \quad \text{O.K.}$$

* Supporting beam width assumed 12″.

The bond stress is practically within the allowable even when the longitudinal slab steel and the stiffening effect of the adjoining slabs are neglected. Accordingly, the **bond stress** on the slab steel is **seldom investigated in** the ordinary **slab design.**

Illustrative Prob. 105d. Design a slab to carry a L.L. of 150#/□′ and to span 12′-0″. Slab is fully continuous. Use Boston Law.

L.L.	= 150#/□′	When $f_c = 0.325 \times 2200 = 715\#/\square''$,
1″ Grano.	= 12	and $f_s = 18{,}000\#/\square''$, $K = 117$
7″ Slab	= 87	
Plaster	= 5	
T.L.	= 254 say 255	

$$M = 1.0 \cdot w \cdot L^2 = 1.0 \times 255 \times (12)^2 = 36{,}800''\#$$

$$d = \sqrt{\frac{36{,}800}{117 \times 12}} = 5.13.$$

Here is an instance where a redesign can reduce the amount of concrete.

L.L.	= 150#/□′	$M = 1.0 \times 242 \times (12)^2 = 34{,}800''\#$
1″ Grano.	= 12	$d = \sqrt{\dfrac{34{,}800}{117 \times 12}} = 4.98''$
6″ Slab	= 75	
Plaster	= 5	Use 6″ slab.
T.L.	= 242	$d = 5.0''$

$$A_s = \frac{M}{f_s \cdot j \cdot d} = \frac{34{,}800}{18{,}000 \times \frac{7}{8} \times 5} = 0.44\square''$$

$$\frac{0.44}{0.20} = 2.2 \qquad \frac{12}{2.2} = 5.48 \qquad \text{Use } \tfrac{1}{2}''\ \phi - 8''\text{ o.c.}$$

$$\frac{0.44}{0.30} = 1.48 \qquad \frac{12}{1.48} = 8.06 \qquad \tfrac{3}{8}''\ \phi \text{ spacers} - 24''\text{ o.c.}$$

Prob. 105e. Design a slab to carry a load of 100#/□′ and to span 11′-0″ if it is partially continuous $\left(M = \frac{w \cdot L^2}{10}\right)$. Use $f_c = 650\#/\square''$ and $f_s = 16{,}000\#/\square''$. 1″ granolithic finish floor and ceiling plastered direct.

Prob. 105f. Design the typical interior slab for the following conditions:

L.L.	= 60	Span 8′-0″
$\frac{7}{8}''$ Fin. Flr.	= 3	Use Boston Law
$\frac{7}{8}''$ Sub. Flr.	= 3	
2″ Cinder Concr. Fill	= 16	
Slab	= —	
Susp. Ceil.	= 15	
T.L.	= —#/□′.	

106. Other Forms of Slab Reinforcement.

Reinforcement for concrete slabs is commonly supplied in the form of round rods and occasionally square bars, but in many instances cold drawn steel wire fabric or wire mesh † is used for slab spans up to 14′-0″ as a limit.‡

† See "Tentative Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement," Serial Designation: A82-21T, American Society of Testing Materials.

‡ Such reinforcement is also used for light walls.

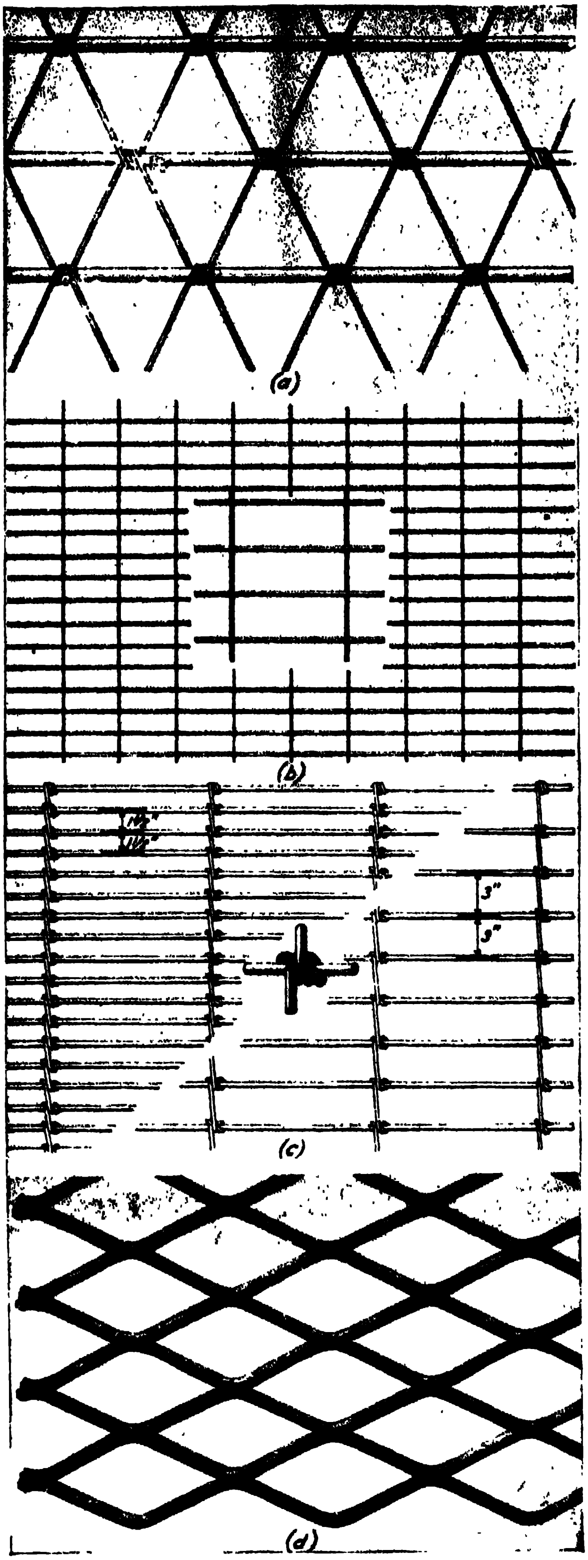


Fig. 159. Concrete Reinforcement

(a) Δ-Mesh (c) Lock-Woven
(b) Welded Wire (d) G. F. Expanded Metal

SPECIFICATION CLAUSE*

Maximum Spans — **Reinforcing materials which are self-centering shall not be used in spans exceeding eight feet. Fireproofing under self-centering reinforcement may be of Portland cement plaster.**

Wire fabric, in place of the ordinary form of bar reinforcement, is finding a great place in the concrete industry. It does away with the handling of many pieces, and the incidental tying together of the same, is readily bent to curves, but has the disadvantage of decreased flexibility of design. In general this material is made up of heavy wires called **carrier** wires crossed by lighter ones, called **tie** wires. In this way the reinforcing maintains its uniform spacing. Among the most common kinds are:— **Welded Wire Fabric,** manufactured by the Clinton Wire Cloth Company, **Triangle Mesh Wire Fabric,** manufactured by the American Steel and Wire Company, **Unit Wire Fabric,** furnished by the American System of Reinforcing, and **Lock-Woven Steel Fabric,** manufactured by the Page Steel and Wire Company. Figure 159 illustrates these types. Table 37 gives condensed data on these materials taken from the catalogues of the manufacturers. It will be noted that the style number of Triangle Mesh Fabric corresponds to the sectional area in square inches per foot of width, while the fabric number of Lock-Woven Steel Fabric is an index to the gauge of the longitudinal wires and if prefixed by 2 or 3 it indicates the number of wires in each longitudinal.

TABLE 37

DATA ON WIRE FABRIC

TRIANGLE (Δ) MESH FABRIC

Longitudinals spaced 4 ins.; cross wires No. 14 gauge spaced 4 ins.

Style number	Number and gauge of wires, each longitudinal, American Steel & Wire Company's steel wire gauge	Sectional area longitudinals, sq. in. per foot width	Total effective longitudinal sectional area, sq. in. per foot width	Approximate weight lbs. per 100 sq. ft.
032	1 — No. 12 gauge	0.026	0.032	22
040	1 — " 11 "	0.034	0.040	25
049	1 — " 10 "	0.043	0.049	28
058	1 — " 9 "	0.052	0.058	32
068	1 — " 8 "	0.062	0.068	35
080	1 — " 7 "	0.074	0.080	40
093	1 — " 6 "	0.087	0.093	45
107	1 — " 5 "	0.101	0.107	50
126	1 — " 4 "	0.120	0.126	57
146	1 — " 3 "	0.140	0.146	65
153	1 — " ¼ inch	0.147	0.153	68
168	1 — No. 2 gauge	0.162	0.168	74
180	2 — " 6 "	0.174	0.180	78
208	2 — " 5 "	0.202	0.208	89
245	2 — " 4 "	0.239	0.245	103
267	3 — " 6 "	0.261	0.267	111
287	3 — " 5½ "	0.281	0.287	119
309	3 — " 5 "	0.303	0.309	128
336	3 — " 4½ "	0.330	0.336	138
365	3 — " 4 "	0.359	0.365	149
395	3 — " 3½ "	0.389	0.395	160

* The Building Law of the City of Boston.

TABLE 37 — *Continued*

Longitudinals spaced 4 ins.; cross wires No. 12½ gauge spaced 8 ins.

Style number	Number and gauge of wires, each longitudinal, American Steel & Wire Company's steel wire gauge	Effective sectional area of cross reinforcement, sq. in. per foot width	Effective longitudinal sectional area, sq. in. per foot width	Approximate weight lbs. per 100 sq. ft.
[illegible]	1 — No. 12 gauge	0.014	0.041	21
049R	1 — " 11 "	0.014	0.049	24
058R	1 — " 10 "	0.014	0.058	28
067R	1 — " 9 "	0.014	0.067	31
077R	1 — " 8 "	0.014	0.077	35
089R	1 — " 7 "	0.014	0.089	40
102R	1 — " 6 "	0.014	0.102	44

150, 200 and 300 foot rolls; 16, 20, 24, 28, 32, 36, 40, 44, 48, 52 and 56 inches wide.

WELDED WIRE FABRIC

Gauge of longitudinal wires, W. & M. gauge	Diameter of longitudinal wires, in.	Area of one longitudinal wire, sq. in.	Gauge of transverse wires, W. & M. gauge	Spacing of transverse wires, in.	Area per foot of width in longitudinal wires only — Spacing of longitudinal wires: 2 in.	3 in.	4 in.	5 in.	6 in.
0	0.307	0.074	6	16	0.443	0.295	0.221	0.177	0.148
1	0.283	0.063	6	16	0.377	0.252	0.189	0.151	0.126
2	0.263	0.054	8	16	0.325	0.217	0.162	0.130	0.108
3	0.244	0.047	8	16	0.280	0.187	0.140	0.112	0.093
4	0.225	0 040	9	16	0.239	0.160	0.120	0.096	0.080
5	0.207	0.034	9	16	0.202	0.135	0.101	0.081	0.067
6	0.192	0.029	10	16	0.174	0.116	0.087	0.069	0.058
7	0.177	0.025	10	16	0.148	0.098	0.074	0.059	0.049
8	0.162	0.021	10	12	0.124	0.082	0.062	0.049	0.041
9	0.148	0.017	11	12	0.104	0.069	0.052	0.041	0.035
10	0.135	0.014	12	12	0.086	0.057	0.043	0.034	0.029

Longitudinal wires: Spacing, centers	Size	Transverse wires: Spacing, centers	Size	Based on longitudinal wires only in 1 foot of fabric width: Sectional area, sq. in.	Description of rolls: Length, feet	Width, in.	Weight, lbs.
Inches	No.	Inches	No.				
2	3	16	8	0.2798	150	62	777
3	3	16	8	0.1865	150	86	746
3	4	16	9	0.1594	150	86	636
4	3	16	8	0.1399	150	86	579
3	5	16	9	0.1346	150	86	545
3	6	16	10	0.1158	200	86	623
4	5	16	9	0.1009	150	86	425
3	7	16	10	0.0984	200	86	537
4	6	16	10	0.0868	200	86	486
3	8	12	10	0.0824	200	86	473
3	8	8	10	0.0824	200	86	509
4	7	16	10	0.0737	200	86	420
3	9	12	11	0.0690	200	86	395
4	8	12	10	0.0618	200	86	378
4	8	8	10	0.0618	200	86	411
4	9	12	11	0.0518	200	86	318
5	12	9	12	0.0209	400	102	385

UNIT WIRE FABRIC

Gauge of carrying wires	Gauge of cross wires	Distance center to center: Carrying wires	Cross wires	Sectional area in square inches per foot width
		Inches	Inches	
11	11	6	6	0.023
10	10	6	6	0.028
9	11	6	6	0.035
9	11	4	12	0.05
9	11	3	12	0.07
8	11	4	12	0.062
7	11	4	12	0.074
6	11	4	12	0.087
5	11	4	12	0.10
4	11	4	12	0.12
3	11	4	12	0.14

LOCK WOVEN STEEL FABRIC

All longitudinals spaced 3 ins.; transverse wires, 12 ins.

Fabric No.	No. wires each longitudinal	Size wires in each longitudinal	Sectional area each longitudinal	Gauge cross wire	Area of cross section per foot width of fabric, sq. in.	Weight per 100 square feet
14 A	1	14	0.0050	14	0.0201	11.04
13 A	1	13	0.0066	14	·0.0263	12.91
12 A	1	12	0.0088	14	0.0350	15.85
12 B	1	12	0.0088	12	0.0350	18.44
11 A	1	11	0.0114	14	0.0456	17.47
11 B	1	11	0.0114	12	0.0456	20.34
9½A	1	9½	0.0156	14	0.0624	27.72
9½B	1	9½	0.0156	12	0.0624	30.75
9 A	1	9	0.0173	14	0.0691	28.60
9 B	1	9	0 0173	12	0.0691	31.78
29 A	2	9	0.0346	14	0.1382	55.58
29 B	2	9	0 0346	12	0.1382	59.60
39 A	3	9	0 0516	14	0.2064	81.95
39 B	3	9	0.0516	12	0.2064	87.46
7½A	1	7½	0.0226	14	0.0904	37.50
7½B	1	7½	0 0226	12	0.0904	40.42
27½A	2	7½	0.0452	14	0.1809	70.88
27½B	2	7½	0.0452	12	0.1809	76.04
37½A	3	7½	0.0678	14	0.2713	105.19
37½B	3	7½	0.0678	12	0.2713	111.10
7 A	1	7	0.0246	14	0.0984	39.48
7 B	1	7	0.0246	12	0.0984	42.84
27 A	2	7	0.0492	14	0.1969	77.54
27 B	2	7	0 0492	12	0.1969	82.32
37 A	3	7	0.0738	14	0.2953	114.32
37 B	3	7	0.0738	12	0.2953	120.51
23 B	2	3	0 0935	12	0.3740	142.88

Regular rolls 150 and 300 feet in length.
Made in widths of any multiple of 3 inches from 18 to 54 inches.

Illustrative Prob. 106a. If a cross sectional area of 0.07□" were specified, what wire fabric would be used of each of the types given in Table 37?

Triangle Mesh #080
Welded Wire Fabric 9–11–3"
Unit Wire Fabric 9–11–3 × 12
Lock-Woven Steel #9A.

Illustrative Prob. 106b. If a 6" slab were reinforced with Triangle Mesh, #365, and it were required to substitute Lock-Woven Fabric, what specification could be used?

Δ mesh, #365 = 0.365□"
From Table 37, Use L.W.S.F. #23B (A_s = 0.374□")
or 2 layers of #27½A.

Cold-drawn steel which is in the form of a mesh may also be used for slab reinforcement. This is made from standard gauge metal which is punched, pressed and expanded by special machines. The shapes of the meshes vary, some utilizing the principle of two-way reinforcement (Art. 107) by means of a diamond mesh, while others depend upon the stamped rib for reinforcement, the intermediate parts being used as spacers and ties only. Figure 160 shows some of the common types and Table 38 gives data for them, taken from the manufacturers' catalogues.

TABLE 38

DATA ON WIRE MESH

Name	Manufacturer	Size of sheet: Width	Size of sheet: Length	Sheets per bundle or crate	Square feet per bundle or crate	Gauges	Sectional area as reinforcement
EUREKA	Northwestern Expanded Metal Co.	21″	8″	9	126	26	
		28″			132	24	
						22	
T-RIB CHANELATH	"	4″ to 48″ by 4″ intervals	By ft. from 3′ to 12′	No. ordered crated		24	0 244
						26	0.183
						28	0 152
SELF-SENTERING	General Fireproofing Co.	20″	4′	12	116	All Lengths in 24 26 and 28	0.277
			5′		145		0.208
			6′		174		0.173
			7′		203		
			8′		232		
			9′		261		
			10′	10	242 / 290		
			11′	8 / 10	213 / 266 / 319		
			12′		232 / 290 / 348		
				24 / 26 / 28	24 / 26 / 28		
				Gauges	Gauges		

Name	Manufacturer	Size of sheet: Width	Size of sheet: Length	Sheets per bundle or crate	Square feet per bundle or crate: 8′	10′	12′	16′	Gauges	Sectional area as reinforcement
STEELCRETE	Consolidated Expanded Metal Co.	6′ 0″	8′ 0″ 12′ 0″ and 16′ 0″ No. 9 gauge 10′ 0″ also	10	480		720	960	13	0 075
		6′ 9″		7	378		568	757		0 10
		5′ 3″			295		442	590		0 125
		7′ 0″		5	280	350	420	560	9	0 15
		6′ 0″			240	300	360	480		0 175
		5′ 3″			211	263	316	421		0 20
		4′ 0″			160	200	240	320		0 25
		7′ 0″		2	112	140	168	224		0 30
		6′ 0″			96	120	144	192		0 35
		7′ 0″			112		168	224	6	0.40
		6′ 3″			100		150	200		0.45
		5′ 9″			92		138	184		0 50
		5′ 3″			84		126	168		0 55
		4′ 9″			76		114	152		0 60
		5′ 9″		1	46		69	92	1	0 75
		4′ 3″			34		51	68		1 00

Illustrative Prob. 106c. Specify the Steelcrete, T-rib, Chanelath, Self-Sentering and G F Expanded wire mesh to give the same area of reinforcement in a 6″ slab as ½″ ϕ rods 10″ o.c. give.

½″ ϕ = 0.196□″ $\qquad 0.196 \times \frac{1[illegible]}{10} = 0.2$[illegible]

Use # 9 Ga. –0.25 Steelcrete, or
#24 Ga. T R Chanelath, or
#24 Ga. Self-Sentering, or
#10 Ga. –0.265 G F Expanded Metal.

Prob. 106d. If a 4″ slab were reinforced with Welded Wire Fabric, gauge 3–8, 2″ spacing, and the Building Law called for 0.6 of 1% of steel in this case, would you [illegible] the slab, if you were the building inspector?

Prob. 106e. If you have Unit Wire Fabric of 6–11 gauge delivered to a job to be incorporated into a 4″ slab, what percentage of reinforcement occurs?

Prob. 106f. How would you specify the substitution of ½″ ϕ rods to replace 0–6, 2″ spacing, Welded Wire Fabric in a 6″ slab?

Prob. 106g. What is the cross sectional area of the Unit Wire Fabric of nominal gauge 6–11? Of Welded Wire Fabric nominal gauge 5–9, 4″ spacing? Of Triangle Mesh, #168? Of Lock Woven Fabric, #37½B?

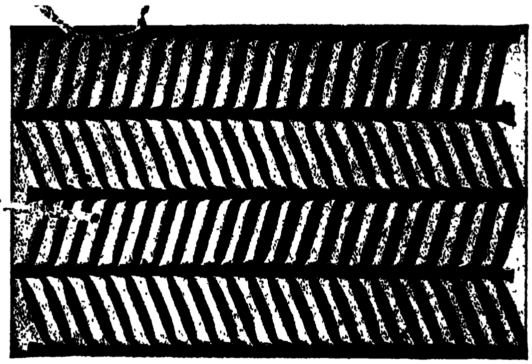
Prob. 106h. If a slab were detailed for ½″ ϕ rods, 6″ o.c., what size of Steelcrete could be substituted? Of G F Expanded Metal?

107. Slabs with Two-way Reinforcement.*

If a slab panel is square, or nearly so, it may be reinforced in two directions so that a saving may be effected. Unless there is a considerable number of such panels the saving may not be large enough to warrant the use of this type of slab. Especially when there are one-way slabs also, confusion may result when the steel is placed. Theoretically, the two-way slab is good design. Some rods are always placed longitudinally in a one-way slab to act as spacers and temperature steel, and by putting such steel at a closer spacing and making it a little heavier, a reduction may be made in the transverse reinforcement, as well as in the thickness of slab.

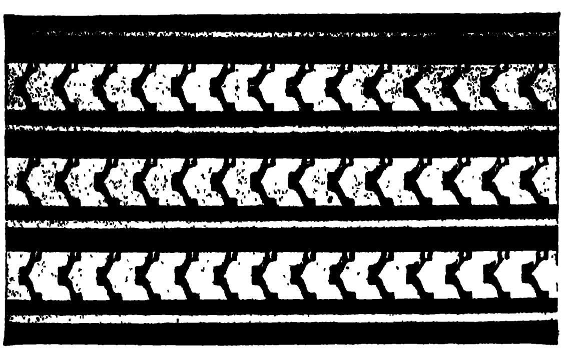
An exact analysis of the moment to be carried by each system of reinforcement cannot be made as the problem is indeterminate due to the more or less flexible supports. Approximate solutions are based on the assumption that the load at any point is carried by the two systems in proportion to the stiffness of the beam elements in the corresponding directions. The bending moment at mid-span then is slightly greater than one-half the moment carried by a slab reinforced in one direction only. The distribution of load varies approximately as a

* For a discussion of the loads brought to beams and girders by two-way slab construction, see Art. 145.

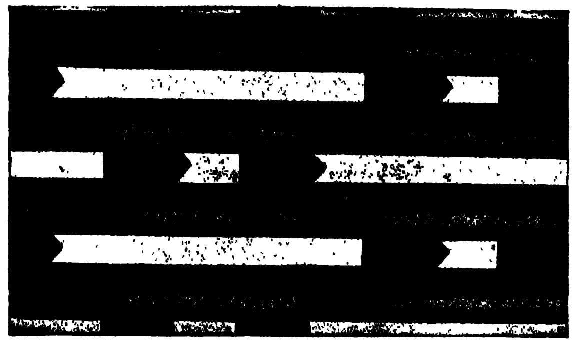


(*a*) GF Herringbone — flat expanded

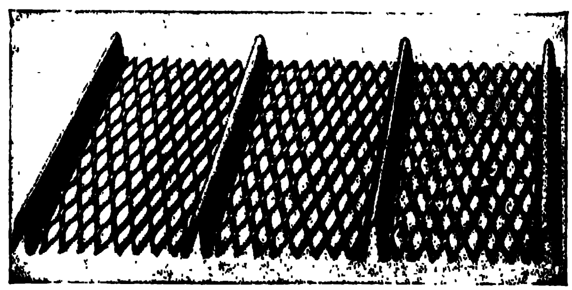
(*b*) Berger — $\frac{3}{8}$″ Ribplex — rib expanded



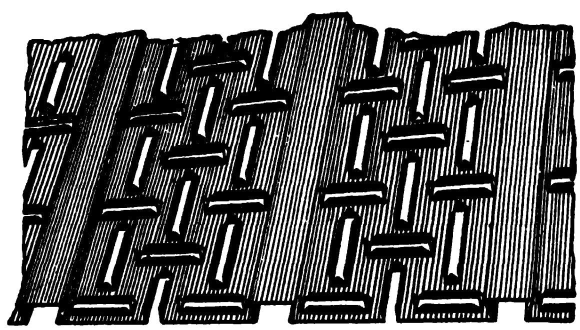
(*c*) Truscon — $\frac{1}{8}$″ — 1A Rib lath



(*d*) Goldsmith — clincher — sheet lath



(*e*) Self Sentering — $\frac{3}{4}$″ Deep rib lath



(*f*) Goldsmith — Shurebond — Deep rib sheet lath

FIG. 160. METAL LATHS*

* See Table 38.

parabola, as indicated in Fig. 161, and should be based upon equal deflections.

The slab should be designed and reinforced as continuous over the supports in order to make it act according to the assumptions.

When the panels are square, the obvious assumption made in practice is that the **beam elements in each direction take one-half of the load.** The error

TABLE 38 — *Continued*

DATA ON WIRE MESH

Name	Manufacturer		Size of sheet: Width	Size of sheet: Length	Sheets per bundle or crate	Square feet per bundle or crate	Gauges	Sectional area as reinforcement
CORR-MESH	Corrugated Bar Co.	$\frac{5}{16}$" rib 3" o.c.	18"	8'	100	1200	24 26 28 in both widths	
				12'		1800		
		⅜" rib 3¼" o.c.	13"	6'		650		
				8'		867		
				10'		1083		
				12'		1300		
TRUSSIT	General Fireproofing Co.		19"	8'	10	126	24, 26, or 27	
				10'		158		
				12'		189		
HERRINGBONE BB	General Fireproofing Co.		20¼"	8'	15	204	22, 24, 26 or 27	
HYRIB	Truscon Steel Co.	1½"	14"	6'	8	56 74 92 112	24	0.236
							26	0.177
		⅞"	24"	8'		96 128 160 192	24	0.219
							26	0.184
							28	0.137
		⅞"	16"	10' or 12'	16	128 171 213 256	24, 26, and 28	
		⅜"	20"			160 212 266 320		
		Rib lath	21"	8'	12	168		
		Rib				6' 8' 10' 12'		

Name	Manufacturer	Size of sheet: Width	Size of sheet: Length	Sheets per bundle	Square feet per bundle	Gauges	Sectional area as reinforcement
RIBPLEX	Berger Manufacturing Co.	24"	By ft. from 4' 0" to 12' 0"	Per order		24	0.198
						26	0.148
						28	0.124
						All lengths	
GF EXPANDED METAL	General Fireproofing Co.	3' 0"	6'	Per order		10	0.176
		4' 0"	8'				
		5' 0"	9'				
		6' 0"	and				
		4' 0"	10' 8"			10	0.265
		5' 4"					
		3' 0"				10	0.353
		4' 0"				and	
		6' 0"				12	0.150
		3' 0"	6'			12	0.194
		4' 0"	and				
		6' 0"	8'			12	0.246

introduced by such an assumption is slight and it is offset by the fact that the designing load is only approximate. The moment for a one-way slab fully continuous is usually calculated from $M =$

Similarly the moment for the same conditions for a square two-way slab should be calculated from $M = \frac{w \cdot L^2}{24}$. The theoretical depth of slab for the latter should be that to the center of the upper layer of rods. The concrete has to sustain the compression due to the moment in one direction only as the stresses are at right angles to each other. The stresses in the concrete in one direction do not weaken the concrete with respect to the stresses in the other direction. The area of steel required in each direction may then theoretically be

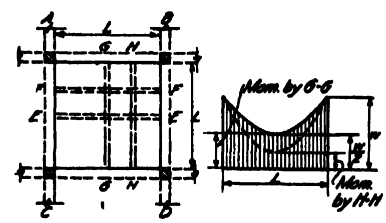


Fig. 161

calculated on the basis of the depth to the layer in question. Practically, the effective depth used is to the center of the upper layer of steel. Both sets of steel may then be spaced the same. Theoretically the spacing of the rods in the lower layer might be increased slightly as they are acting at a larger effective depth, but the increase in spacing gained would not warrant destroying the advantage of symmetry in the two directions.

Since the distribution of load is assumed to vary as a parabola, the bending moment is greater near the center of the slab than near the edges, and the reinforcement may be arranged accordingly.

SPECIFICATION CLAUSE*

In placing reinforcement in such slabs account shall be taken of the fact that the bending moment is greater near the center of the slab than near the edges, and two-thirds of the calculated moments shall be assumed as carried by the center half of the slab and one-third by the outside quarters.

Some designers in following this specification prefer to calculate the moment in one direction for the whole panel and the required area of steel, and the number of rods corresponding. Then two-thirds of this number are spaced equally in the middle half of the span and

* The Building Law of the City of Boston.

the remainder apportioned to the quarter portions either side. If the panels are small, the weight of steel saved by varying the spacing may not compensate for the extra labor, inconvenience, and possible mistakes in placing it. Consequently for small panels, it may be advisable to space the rods uniformly across the span in both directions.

Illustrative Prob. 107a. Design a two-way slab for a typical interior panel 20′-0″ square to carry a L.L. of 60#/□′. Boston [illegible]

L.L. =	60
⅞″ Fin. Flr. =	3
⅞″ Sub. Flr. =	3
2″ Cinder Concr. Fill =	16
7″ Slab =	88
Susp. Ceil. =	15
T.L. =	185#/□′

$$M = \frac{w \cdot L^2}{24} = 0.5\, w \cdot L^2 \text{ in.-lbs.}$$

$$M = 0.5 \times 185 \times (20)^2 = 37{,}000''\#$$

$$d = \sqrt{\frac{37{,}000}{117 \times 12}} = 5.1+''$$

Use 6½″ slab on a/c of two layers of steel.

M for whole panel in one direction

$$37{,}000''\#/\text{ft.} \times 20 = 740{,}000''\# \quad \text{Ave. } d = 5.2''$$

$$A_s = \frac{740{,}000}{18{,}000 \times \frac{7}{8} \times 5.2} = 8.83\square''.$$

Try ⅝″ ϕ rods, area = 0.30□″ $\qquad \dfrac{8.83}{0.30} = 29+$, say 30 rods.

$\frac{2}{3} \times 30 = 20$ in middle 10′-0″

$\dfrac{10 \times 12}{20} = 6''$ o.c. $\qquad$ Use ⅝″ ϕ rods 6″ o.c. for middle 10′-0″ and ⅝″ ϕ — 12″ o.c., for two 5′-0″ outside portions — both ways.

When the slab panel is oblong, the proportion of load carried by each beam element must be calculated. In Fig. 162, let

w_0 = the total load in #/□′ on the slab,
w_L = the proportion of load carried in the longitudinal direction, and
w_T = the proportion of load carried in the transverse direction.

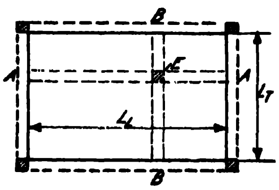


FIG. 162

The deflections of any two strips of unit width such as A–A and B–B must be the same at their point of intersection k.* The deflections are proportional to the 4th power of the lengths in the respective directions. Thus

$$\frac{w_T}{w_L} = \frac{L_L^4}{L_B^4}.$$

But $w_0 = w_L + w_T$, or $w_L = w_0 - w_T$.

* It is obvious that the shorter span will take the larger proportion of the total load as it is stiffer and resists bending to a greater extent. In other words, beams take load in proportion to their stiffness.

Substituting,

$$\frac{w_T}{w_0 - w_T} = \frac{L_L^4}{L_B^4}.$$

Expanding, $w_T \cdot L_B^4 = w_0 \cdot L_L^4 - w_T \cdot L_L^4$.
Collecting terms, $w_T\,(L_B^4 + L_L^4) = w_0 \cdot L_L^4$.

$$\frac{w_T}{w_0} = \frac{L_L^4}{L_B^4 + L_L^4}.$$

The following gives the ratios of w_T to w_0 for varying ratios of L_L to L_B.

Ratio $\frac{L_L}{L_B}$	1.0	1.1	1.2	1.3	1.4	1.5	2.0
$\frac{w_T}{w_0}$	0.50	0.59	0.67	0.75	0.80	0.83	0.89

A simpler method of expressing the proportion of the total load carried by the transverse reinforcement, namely,

$$r = \frac{w_T}{w_0},$$

is usually used as is specified below.

SPECIFICATION CLAUSE †

For oblong slabs, the length of which is not greater than one and one-half times their width, the moment to be resisted by the transverse reinforcement may be found by using a proportion of the live and dead load equal to that given by the formula $r = \frac{l}{b} - 0.5$, where l = length and b = breadth of slab. The longitudinal reinforcement should then be proportioned to carry the remainder of the load.

This practically agrees with the previous table for when

$$\frac{L_L}{L_B} = 1.1, \qquad \frac{w_T}{w_0} = 0.59.$$

Correspondingly, $r = 1.1 - 0.5 = 0.6$, and so on. To maintain consistent nomenclature the formula may then be expressed as

$$r = \frac{L_L}{L_B} - 0.5.$$

When $\frac{L_L}{L_B}$ becomes 1.5, according to the specification, $r = 1.5 - 0.5 = 1.0$. This would mean that the transverse reinforcement carries all the load according to the formula, and that the longitudinal reinforcement is carrying no load. Consequently,

† From Joint Committee Report of 1916.

when the length of the oblong slab is greater than 1.5 times the width, a two-way system of reinforcement is of no advantage. As the ratio r approaches 1.5, the value of the longitudinal steel in carrying load becomes rapidly less.

The first test as to whether a two-way slab will serve to any advantage or not is to calculate the ratio of the length of the panel to the width. A good practical limit for the ratio based upon experience is 1.2. When it is feasible to try a two-way slab, both a one-way and a two-way design should be made and the results compared as to cost. In a one-way slab, some longitudinal steel is used as spacers and the amount of such reinforcement in a two-way slab may not be much more. In such a case a two-way slab would be economical as the amount of concrete required would be enough less to more than offset the extra steel.* The larger bending moment is based upon the short span and this moment controls the depth of the slab. Calculations should be made to verify the fact that the longitudinal steel, when placed over the transverse steel, will carry its proportion of load safely.

Illustrative Prob. 107b. Design a two-way slab for a typical interior panel, 12'-0" × 14'-0" for a live load of 300#/□'. Use f_c = 650#/□" and f_s = 16,000#/□". Design the panel for a one-way slab and compare the costs.

$$L_L = 14.0,\ L_B = 12.0,\ \frac{L_L}{L_B} = \frac{14}{12} - 0.5 = 0.67 \text{ by 12'-0'' span.}$$

0.33 by 14'-0" span.

L.L. = 300 $M = 1.0\,w \cdot L^2$ (both directions)
1" Grano. = 12
6" Slab = 75
T.L. = 387#/□'

$$M = 1.0 \times (0.67 \times 387) \times (12)^2 = 37{,}400''\#$$

$$d = \sqrt{\frac{37{,}400}{107.4 \times 12}} = 5.4''$$

Use 6½" slab.
d = 5.5"

$$A_s = \frac{37{,}400}{16{,}000 \times \tfrac{7}{8} \times 5.5} = 0.49□''$$

½" ϕ, area = 0.196 $\frac{0.49}{0.196} = 2.54$ $\frac{12}{2.54} = 4.6$

⅝" ϕ, area = 0.307 $\frac{0.49}{0.307} = 1.6$ $\frac{12}{1.6} = 7.5$

Use ⅝" ϕ — 7½" o.c. in 12'-0" direction.

The depth of the slab is controlled by the larger moment, but it must be shown that the long span steel may be placed above the short span steel with safety.

If ⅝" ϕ rods are used in both layers, as is desirable, to keep a minimum number of sizes of rods, the effective depth to the upper layer is 5.5 − 0.625 = 4.88".

$$M = 1.0 \times (0.33 \times 387) \times (14)^2 = 25{,}000''\#$$

$$d\ (\text{required}) = \sqrt{\frac{25{,}000}{107.4 \times 12}} = 4.4''\quad \text{O.K.}$$

$$d\ (\text{actual, as above}) = 4.88''$$

$$A_s = \frac{25{,}000}{16{,}000 \times \tfrac{7}{8} \times 4.88} = 0.368$$

$\frac{0.368}{0.307} = 1.19$ $\frac{12}{1.19} = 10.1$

Use ⅝" ϕ — 10" o.c. in 14'-0" direction.

The spacing of the rods in both directions might have been varied similar to that shown in Illustrative Prob. 107a, but the spacing will be kept uniform on account of the size of the panel.

One-way slab.

L.L. = 300 $M = 1.0 \times 412 \times (12)^2 = 59{,}300''\#$
1" Grano. = 12
8" Slab = 100
T.L. = 412#/□'

$$d = \sqrt{\frac{59{,}300}{107.4 \times 12}} = 6.81$$

Use 8" slab
d = 7.0"

$$A_s = \frac{59{,}300}{16{,}000 \times \tfrac{7}{8} \times 7} = 0.605□''$$

$\frac{0.605}{0.307} = 2.$ $\frac{12}{2} = 6.$

Use ⅝" ϕ — 6" o.c.
Use ⅜" ϕ spacers 24" o.c.

Comparative Cost.

Length of rods in 14'-0" direction = $14 + \frac{14}{3}$ = say 18'-9"

Length of rods in 12'-0" direction = $12 + \frac{12}{3}$ = say 16'-0"

For the two-way panel,

$\frac{12 \times 12}{10}$ = 14.4 spaces, say 16 rods × 18'-9"

$\frac{14 \times 12}{7\frac{1}{2}}$ = 22.4 spaces, say 24 rods × 16'-0"

For the one-way panel,

$\frac{14 \times 12}{6}$ = 28 spaces, say 30 rods × 16'-0"

Spacers, $\frac{12 \times 12}{24}$ = 6 spaces, say 7-⅜" ϕ rods × 14'-0"

Weight of steel, two-way panel ⅝" ϕ = 1.04#/ft. ⅜" ϕ = 0.38#/ft.

16 × 1.04 × 18.75 = 313
24 × 1.04 × 16.0 = 402
715#

Weight of steel, one-way panel

30 × 1.04 × 16.0 = 500
7 × 0.38 × 14.0 = 38
538#

715 − 538 = 177# 177 × \$0.04 = \$7.08

8" slab − 6½" slab = 1½" of concrete

$\frac{14 \times 12 \times 1\frac{1}{2}}{12}$ = 21 c.f. of concrete = 0.78 cu. yd.

\$0.78 × \$14.00 = \$10.92

Extra cost of forms per square foot for heavier slab = \$0.02.

14 × 12 × \$0.02 = \$3.36

Extra cost of placing steel in 2-way panel, \$0.01/#.

177 × \$0.01 = \$1.77

\$7.08	\$10.92	\$14.28
1.77	3.36	8.85
\$8.85	\$14.28	\$5.43

The two-way slab is more economical in this case by \$5.43 per panel. If there were a considerable duplication of panels, this would be an item worth considering. For only a few panels the gain would be offset by the supervision required.

* In checking such calculations it will be safe to assume that a yard of concrete costs about 3.5 times as much as 100 pounds of reinforcement (neglecting saving in forms for 2-way slabs, if any).

Prob. 107c. Design a two-way slab for a typical interior panel 18′-0″ square, to carry a live load of 100#/□′. Use f_c = 650#/□″ and f_s = 16,000#/□″. 1″ granolithic finish and ceiling plastered direct. Vary spacing of rods as allowed.

Prob. 107d. Design a two-way slab for a panel 16′-0″ by 18′-0″. Panel is only partially continuous in the 18′-0″ direction $\left(M = \dfrac{w \cdot L^2}{10}\right)$ and is fully continuous in the 16′-0″ direction. Use Boston Law. Design the panel for a one-way slab and compare the costs.

to the direction of the bars by their spacing, first deducting the halves of the girders on either side. Assuming the girders in Fig. 164 (*a*) to be 14″ wide, the space to be occupied by the bars is (20 × 12) − 14 = 226″. With the bars 10″ o.c., there are 22 spaces with 6″ to spare. Spacing 3″ of this to each side of the girders, 23 bars are needed, or one more than the number of spaces.

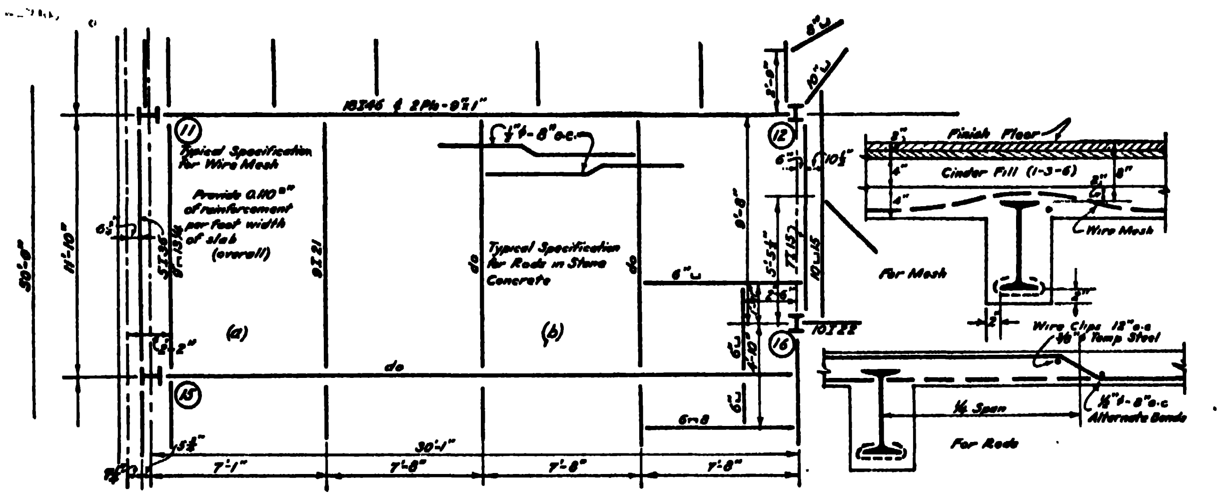


FIG. 163. TYPICAL PLAN INDICATING REINFORCEMENT
(*a*) mesh (*b*) rods

108. Slab Details.

After a slab panel is designed, the detailing incurs the calculation of the number of bars, their lengths and their bendings. Several methods of showing the details for slab steel are used. One is to conventionally indicate the steel directly upon the plan of the panel, as indicated in Fig. 163, which represents a typical engineer's sketch. Another method is to show a cross section through the floor and the relation of the steel to it, as in Fig. 164 (*b*). The latter is a more positive manner of showing all the details. The rods in either case may be separate for each span, or they may be made to continue over several short spans, the former method being preferable. If the latter is used, the rods should break joints. Certain liberties are taken in detailing steel, one of which is to show rods one above the other for clearness, while in reality they all lie in the same plane. Some engineers indicate the steel by dotted lines while others use full lines. It is recommended that the system shown in Fig. 164 be followed. A particular bar is shown with a full line and the one next to it is shown by a dotted line. This method helps to clarify the intention of how the rods are to be bent and where they stop.

The number of bars in any case is obtained by dividing the length of the panel at right angles

The **angle of bending** slab steel is usually 30° with the horizontal. The center of the bends is commonly at the one-fourth or one-sixth points of

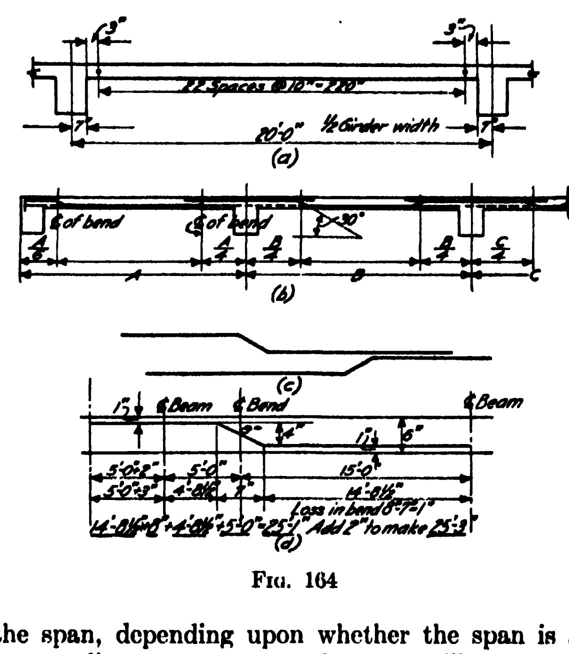


FIG. 164

the span, depending upon whether the span is an intermediate one or an end one, as illustrated in Fig. 164 (*b*). The lap of each bar into the next

panel is also to the one-fourth point. Half of the bars are bent each way as shown in (*c*). Since the bars lap into the adjoining panels and every other rod is bent up, there is as much steel over each support as there is at mid-span. The length of bars is usually given to the nearest 3″ above the calculated value, which is equal to the span plus one-quarter of the adjacent span plus the **loss in bend.** The

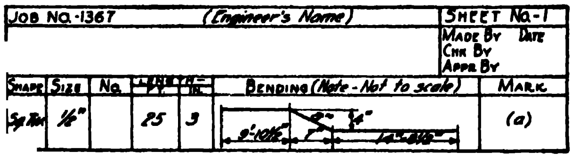


FIG. 165

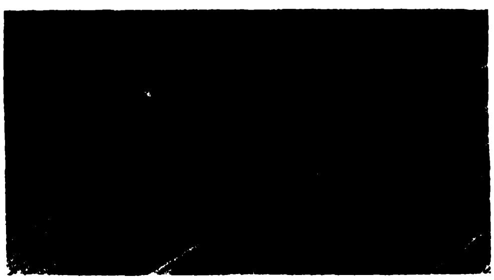
latter is the difference between the slant length of the bent portion and its horizontal projection. Thus in Fig. 164 (*d*),

$(4)^2 + (7)^2 = 65.\quad \sqrt{65} = 8$
Loss in bend $= 8 - 7 = 1''$
$(14'\text{-}8\frac{1}{2}'') + 8'' + (4'\text{-}8\frac{1}{2}'') + 5'\text{-}0'' = 25'\text{-}1''$ theoretical
$(25'\text{-}1'') + 2'' = 25'\text{-}3''$ practical.

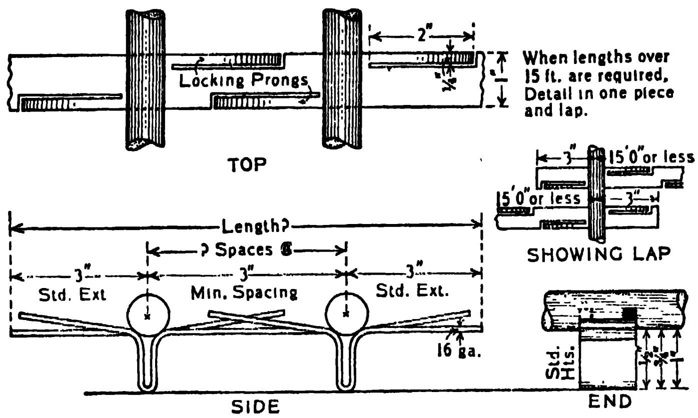
When the ends of the bars are hooked, the common practice is to allow an additional 6″ length for the hook.

In general, engineering details show only the size, spacing, length, bend-points and terminal points of slab rods. All bends are made 30° unless otherwise noted. The structural details will, therefore, be similar to those in Fig. 163 or Fig. 164 (*b*). It is of course evident that more complete information is necessary in order to supply the bars with the correct details. There are two methods commonly employed in such work. Either the contractor buys the steel in proper lengths and has it bent at the job, or he may order the reinforcement bent by the bar company, ready to place in the forms.* In either case, details such as shown in Fig. 164 (*d*), or a **schedule** is required, the latter being the usual procedure. Figure 165 illustrates a common method. The number of like panels is determined from the plan, and with this data, the total number of bars exactly alike can be tabulated and later fabricated. These are assigned a mark and bundles of them are tagged with this identification. If one

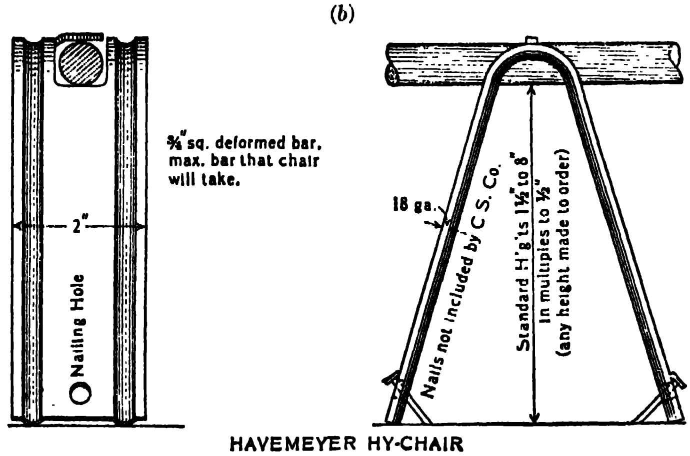
* For a discussion of the methods of bending bars, see Index, Vol. I.



(*a*)



SECURO BAR-SPACER
(*b*)

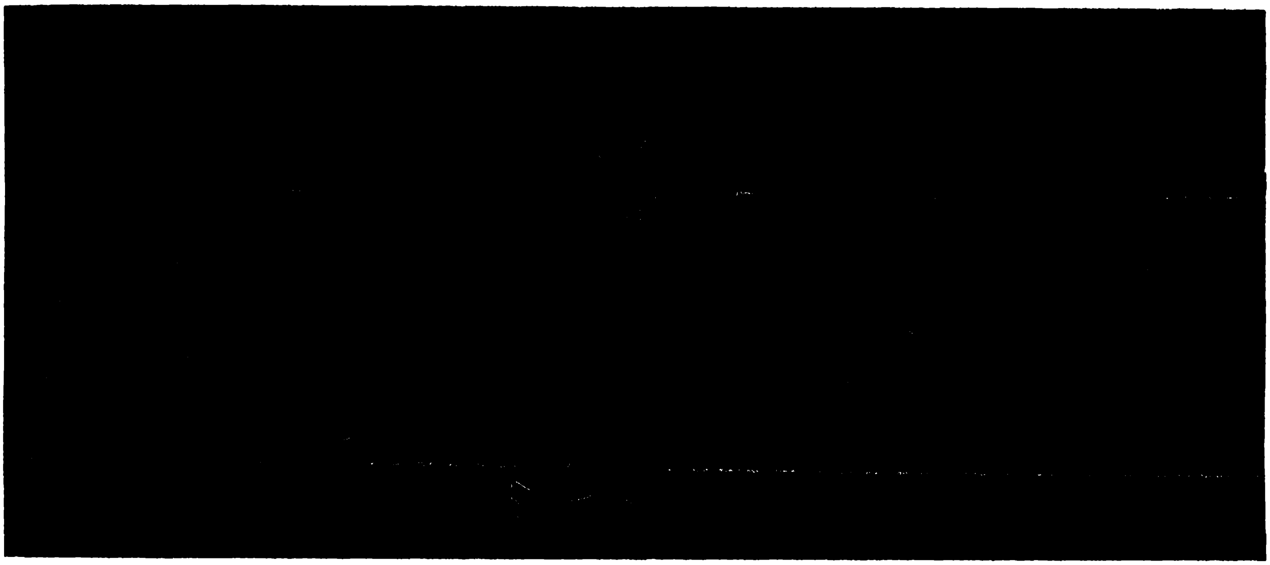


HAVEMEYER HY-CHAIR

(*c*) FIG. 166† (*d*)

† Courtesy of Concrete Steel Company.

(e)



(f)

Fig. 166*

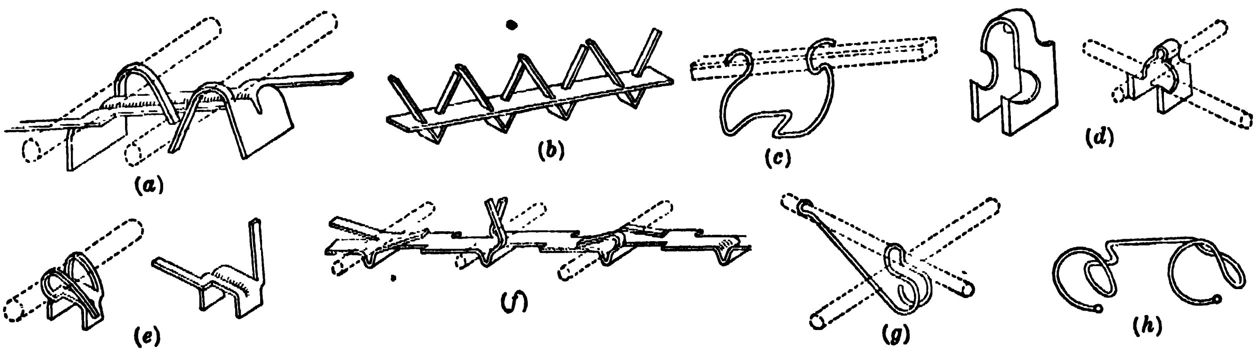


Fig. 167. Slab Bar Ties and Spacers

(a) beam spacers (Universal Form Clamp Co.)
(b) Securo beam spacer (Metal Bldg. Mat. Co.)
(c) bar-chair (Concrete Steel Co.)
(d) chair lock (Electric Welding Co.)
(e) easy chairs (Universal Form Clamp Co.)
(f) Securo slab spacer (Metal Bldg. Mat. Co.)
(g) ty-chair (Concrete Steel Co.)
(h) easel chair (Concrete Steel Co.)

* Courtesy of Concrete Steel Company.

bar differs in any detail from another, it is given an independent mark.

An important feature in connection with steel details is that of properly securing the reinforcement in place, and " a place for every bar and every bar in its place " is a good slogan. If the reinforcement is designed correctly, its effectiveness is marred if it is not in its proper place. Such fastenings should be sufficiently strong, simple in nature, and readily applied, and slipshod methods of using pieces of concrete, brick bats, and chicken wire are not to be recommended. Figure 166 shows some patented forms of bar spacers. That in (*a*) is the Securo, (*b*) shows the Hy-chair form of holding " raising rods," (*c*) the Ty-chair, used at bar intersections, and (*d*) the Bar-Ty. Other types are illustrated in Fig. 167.

On structural plans, each panel is assigned a letter, the latter varying if the panel differs from the others. In this way only the panels which vary need be detailed by cross sections or on the plan; the letters define the remaining panels.

Prob. 106a. Detail a cross section of a slab for the following data:

6″ slab, $\frac{5}{8}''\phi$-6″ o.c. one way
$\frac{3}{8}''\phi$-18″ o.c. temperature steel
Floor beams 12 I 31.8, 8′-0″ o.c.

Show end bay and first interior bay.

PLATE 11 TYPICAL VIEW SHOWING USE OF METAL LUMBER

Courtesy of Berger Manufacturing Company

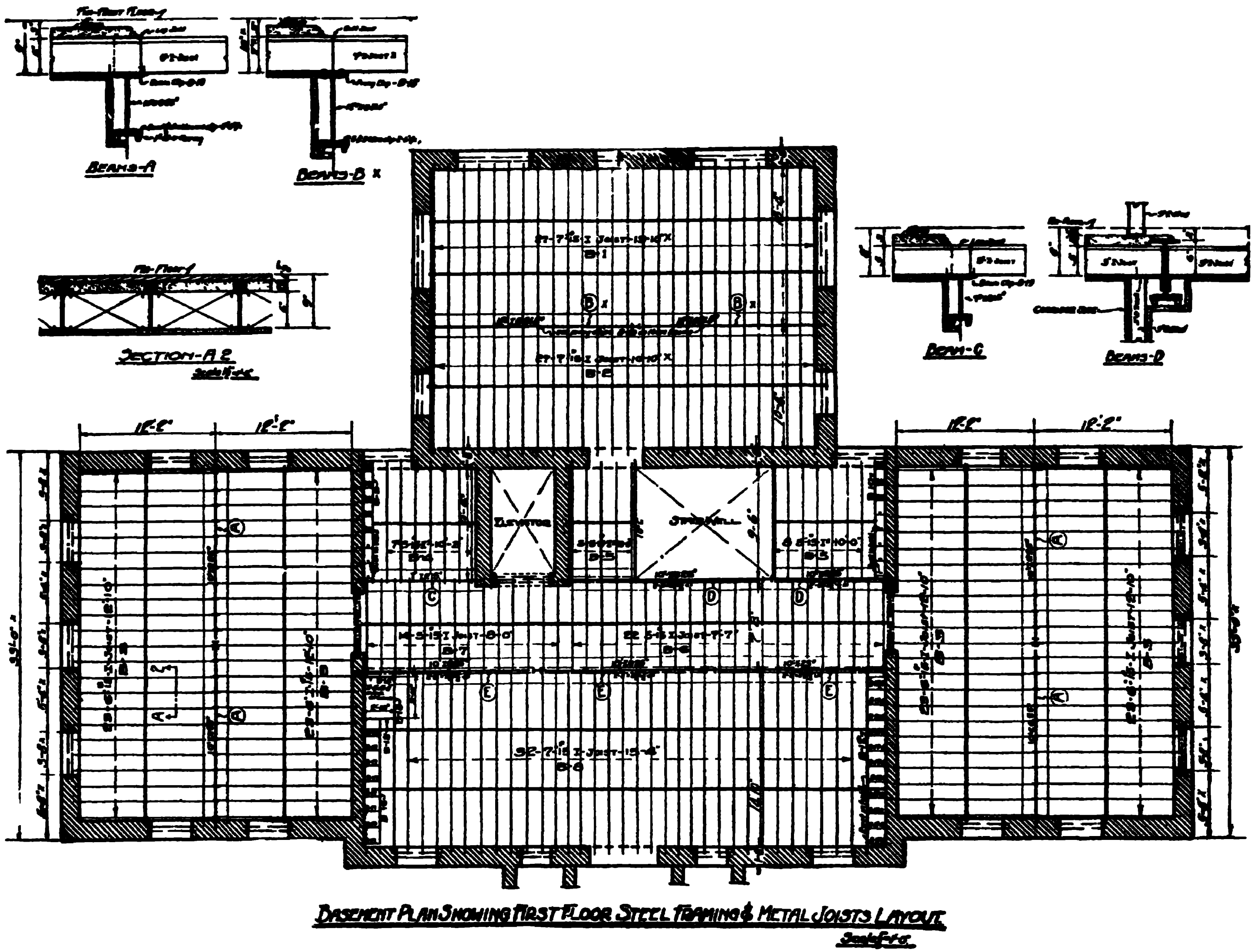


PLATE 12 TYPICAL FRAMING PLAN
METAL LUMBER CONSTRUCTION

Courtesy of Berger Manufacturing Company

CHAPTER 8

JOIST CONSTRUCTION

109. General.

The use of structural steel, rolled and built-up shapes to form the frame for buildings developed principally when it became necessary to erect larger structures than could be built economically with timber. Heavy timber beams and girders occupy too large a proportion of the story heights in many cases and require a larger number of intermediate supports. For buildings of not very great height, masonry bearing walls for exterior supports and steel columns for interior supports, with a floor frame of steel beams and girders, and floor carrying systems of various kinds, are used commonly. Masonry bearing walls for buildings, a large number of stories in height, must, by necessity, be quite thick, particularly in the lower stories, and such walls take up a considerable amount of room, as well as being expensive to build. As a result, the steel **skeleton framed building** has become popular in modern practice for tall buildings. In this, the loads at each floor, including the walls, are carried by the floor frame into both interior and exterior steel columns, and the enclosure is made by the use of curtain walls. Such construction allows a greater speed of erection, less bracing is required, and separate groups of masons may be employed at the same time.

110. Wood Joisted Floors.

The use of wood floor joists supported by steel beams and girders is limited to second class construction naturally, and first floors of apartment houses are sometimes framed in this way although the latest revisions of many important building codes now call for a fireproof first floor, where the fire area exceeds a specified number of square feet. Some even make it a general requirement. The steel girders are used to support bearing partitions which in turn carry the upper floors. This is discussed in Book 1.*

Where a non-inflammable construction is desired, and where the use of a structural concrete slab is not warranted because of its increased cost and weight, metal joists supporting thin concrete slabs and metal lathed, cement plastered ceilings are now frequently used. The cost usually averages about 10% more than a wood-framed floor, but the increased fire protection and other advantages warrant its use, especially for the first floor.

SECTION 8A

METAL LUMBER FLOOR CONSTRUCTION†

111. Typical Construction.

Pressed steel joists are being used in many instances as a substitute for wood joist construction intended for buildings carrying light loads, such as residences, apartment houses, schools, and the like. Among the manufacturers who make this product are the Berger Mfg. Co., Canton, Ohio; the General Fireproofing Co., of Youngstown, Ohio, and the Trussed Concrete Steel Co.‡

Figure 168 shows a typical **section** of a Berger joist, which is virtually two channel pieces, spot welded together, with the toes of the flanges turned in to provide additional stiffness. These joists are made in varying depths from 4″ to 12″ by 1″ increments, and in #11 and #15 gauges which are 0.072″ and 0.090″ in thickness. The metal in each section is all of uniform thickness due to the even rolling, and the sections may be obtained in any reasonable length.§ They

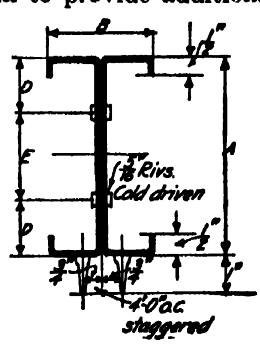


FIG. 168

* Volume II, "Architectural Construction," Book 1, "Wood Construction," by Voss and Varney, — John Wiley & Sons, Inc.

† A new type of steel beam which has recently (1926) come into the market for use in floor construction and as purlins and rafters in roof construction, is the "Junior Beam," an exclusive product of the Jones and Laughlin Steel Corporation, Pittsburgh, Pa. This is a light-weight, rolled-steel, structural section, which is similar in nature to the standard steel beams, and is of structural grade, basic open-hearth steel, rolled from the billet to the full I-section in the typical way. These beams are available in 12″, 11″, 10″, 9″, 8″, 7″ and 6″ depths. Properties of these sections are given in catalogues published by the manufacturers. The beams are used in a manner similar to the metal lumber sections described in Sect. 8A, and standard details may be found in the catalogues furnished by the company.

‡ Trade catalogues may be obtained by applying to any of these companies.

§ As a rule, it is not economical to use joists over 25′-0″ in length. Maximum lengths are subject to shipping and erection limitations, and excessive lengths increase costs. For lengths of special sections, refer to Table 39.

are usually shipped cut to ordered lengths, which saves considerable time in erection, and are usually given a shop coat of paint.

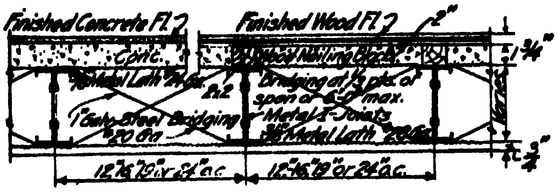


Fig. 169

Figure 169 illustrates the typical construction. Some of the **advantages** claimed are:

(1) The erection is simple and rapid, as the joists are light in weight and may be handled and placed in position by one man.

(2) The dead weight of the floor construction is comparatively small, thus effecting a saving in transportation, equipment and in the sizes of the supporting frame.

(3) No forms are required as the metal lath at the top provides a base for carrying the concrete floor slab, and also serves as reinforcement. The lath can be rolled over the joists, already in place, very readily, so that ease of erection is again served.

(4) The screeds are easily placed as they are nailed with 16d nails driven down in between the webs of the joists and these clinch themselves automatically.

(5) The metal lath for the ceiling is readily attached to the prongs.

(6) The construction has been shown to be fire resisting by the tests for the New York City Building Bureau conducted at the Columbia Fire Testing Station at Brooklyn, N. Y., in April, 1915. The resistance to the action of water pressure and after-loads was also satisfactory.

(7) It is considered vermin-proof and the air spaces provide good heat and sound insulation, as well as room for conduit work.

(8) Easy inspection is possible and the workmen do not require a great amount of previous specialized experience.

The finish flooring may be varied according to the type desired. When a granolithic, terrazzo, or tile surface is desired, the screeds are naturally omitted, but $\frac{3}{8}''\phi$ expansion rods, 18″ o.c., should be provided and placed over the metal lath. The wood finish flooring is often nailed directly to the screeds, but some architects specify a sub-floor as in other types of floor construction. The latter provides additional stiffness and a better and more even nailing surface. Considerable objection is raised to this type of construction on account of the breaking of the concrete at each screed. It would be far better to make the 2″ slab continuous and provide some other means of fastening the wood floor.

112. Properties of I Joists.

The various elements of the metal lumber sections may be calculated in the usual way. The section moduli may be found from the formula:*

$$\frac{I}{c} = A_n \cdot d + \frac{t \cdot d^2}{6}, \text{ in which}$$

A_n = the net flange area (the gross area minus the area of the holes punched out for prongs),

d = the depth of the joist, and

t = the thickness of the web.

It is claimed by some authorities that prongs should not be punched out of the top flange, as tests to destruction show a failure by compression at such places, and that the lath should be fastened by other means at the top flanges. Table 77 gives the common dimensions and properties of standard joists.†

TABLE 39

(BERLOY STANDARD)

Dimensions and Properties of I Joists

Depth (In.)	Weight‡ (Lbs. per Ft.)	Flange Width (In.)	Web§ Thickness (In.)	Area of Section (Sq. Ins.)	Moment of Inertia (Ins.)⁴	Radius of Gyration (In.)	Section Modulus (In.)³
4	3.7	3	0.144	1.08	2.60	1.55	1.30
5	4.2	3	0.144	1.22	4.38	1.89	1.75
6	4.7	3	0.144	1.37	6.90	2.24	2.30
7	5.5	3½	0.144	1.62	11.20	2.63	3.20
8	6.1	4	0.144	1.80	16.80	3.06	4.20
9	7.0	4	0.150	2.06	23.85	3.40	5.30
10	8.0	4½	0.156	2.38	33.25	3.74	6.65
11	9.5	4½	0.172	2.80	46.20	4.06	8.40
12	11.5	4½	0.180	3.10	60.00	4.40	10.00

Tests show the ultimate tensile strength of the material used to vary from 55,000 to 65,000#/□″, so that on the basis of the usual factor of safety, the maximum allowable flexural stress is commonly taken as 16,000#/□″.

* Truscon Concrete Steel Co. (inch units considered).

† The joist sizes and weights have recently been standardized so that all companies manufacture the same sizes.

‡ Subject to 2½% maximum variation.

The properties of channel joists are one-half of those listed.

Special sections, made of steel 0.120″ in thickness, are available. These are used for headers and trimmers and in places where conditions call for greatly increased strength, only. Lengths of such joists from 4″ to 10″ deep should not exceed 16′-8″, and for 11″ and 12″ depths, 12′-6″.

§ The web thickness is the sum of the thicknesses of the two channel webs.

113. Typical Framing Plans.

As for other types of floor construction, typical framing plans are necessary to show the sizes and spacing of the joists, the bridging, the location and sizes of the supporting beams and partitions, column centers, openings, and so on. Plate 12 is a typical framing plan of this kind.

114. Determination of Joist Sizes.

The theoretical calculations to determine the required sizes of joists involve the principles of steel beam design as outlined in Chap. 2. In order to match the standard 96″ length of metal lath, four common spacings of joists are used, namely, 12″, 16″, 19″ and 24″ o.c. The most used of these is the 16″ spacing. Such spacings usually will not cause the intervening concrete to be overstressed.

Illustrative Prob. 114a. Show that the concrete is not overstressed for a condition of heavy loading, in standard construction.

Assume maximum spacing of 24″ o.c.

2-⅞″ Wood Flrs.	= 6	
2″ Concrete	= 25	
D.L.	= 31#/□′	
L.L.	= 100	(assumed heavy)
T.L.	= 131#/□′	

Assume the concrete is simply supported on the clear span between screeds.

For a 1′-0″ strip,

$$M_e = \frac{w \cdot l^2}{8} = \frac{131 \times (24 - 2)^2}{8} = 796''\#$$

$$M_r = \tfrac{1}{2} f_c \cdot k \cdot j \cdot b \cdot d^2$$

Assume d conservatively low for the 2″ thickness, say 1½″

$$796 = \tfrac{1}{2} f_c \times \tfrac{3}{8} \times \tfrac{7}{8} \times 12 \times (1.5)^2$$

f_c (actual) = 180#/□″

f_c (allowable) for 1 : 2½ : 5 concrete = 500#/□″

The condition would be even safer when no screeds are used as the continuity of the concrete would be effective.* Nevertheless the thickness should not be made less than 2″ in any case on account of its fire resisting value. Observations have shown that lath of the proper gauge* will not sag greater than $\frac{1}{24}$ of the center to center of joists with 2″ concrete. This is additional protection, as the thickness of slab is increased correspondingly, for a level top surface is maintained.

When determining the load the joists are to carry, an estimate of the weight of the joists themselves must be made previous to their design. This is most conveniently expressed as so many #/□′ of floor area. Table 40 gives check values for such assumptions.

* The following gauges are recommended by the manufacturers:

Spacing of Joists	Lath for Concrete	Lath for Ceiling
24″	4#	3½#
19	4	3½
16	3½	3
12	3½	3

TABLE 40

APPROXIMATE WEIGHTS OF JOISTS PER SQ. FT. OF FLOOR AREA

Use net area of floor in estimating. Material for laps and bearings is included in weights below.

Linear Ft. of Joists per Sq. Ft. of Floor		1.05	.94	.80	.65	.55
Spacing		12″	13½″	16″	19″ or 20″	24″
Depth of Joist	Weight per Linear Ft.	Weight in Lbs. per Sq. Ft. of Floor Area				
4″	3.7 lb.	3.88	3.48	2.96	2.40	2.04
5″	4.2 lb.	4.41	3.95	3.36	2.73	2.31
6″	4.7 lb.	4.94	4.42	3.76	3.05	2.58
7″	5.5 lb.	5.78	5.17	4.40	3.57	3.02
8″	6.1 lb.	6.40	5.73	4.88	3.96	3.35
9″	7.0 lb.	7.35	6.58	5.60	4.55	3.85
10″	8.0 lb.	8.40	7.52	6.40	5.20	4.40
11″	9.5 lb.	9.97	8.92	7.60	6.17	5.22
12″	10.5 lb.	11.00	9.87	8.40	6.83	5.78

Weights given above are approximate for short cut estimating and checking.

The size of joist required for flexure may be calculated in the usual manner.

Illustrative Prob. 114b. Check the size of joists shown on Pl. 12 for the middle portion of the framing toward the front of the building (panel *B*-8). Live load specified, 50#/□′.

L.L.	= 50	$w = w_0 \times \frac{8}{12}$
Fin. Flr.	= 3	Assume 16″ spacing
Sub Flr.	= 3	$w = 96 \times \frac{16}{12} = 128\#/\text{ft.}$
2″ Concrete	= 25	Span = 14′-10″
Joists	= 5 (Table 40)	
Ceiling	= 10	
T.L.	= 96#/□′	

$$M = 1.5\, w \cdot L^2 = 1.5 \times 128 \times (14.83)^2 = 42{,}200''\#$$

$$\frac{I}{c} = \frac{42{,}200}{16{,}000} = 2.63''^3$$

Use 7″-5.5# Joists, 16″ o.c.

(Refer to Table 39)

Ordinarily, the question of **shear is not investigated** in the design of the joists, as the spans are relatively long and the loads are comparatively light for the cases to which such construction is adaptable. Furthermore, the usual allowable shearing stress for steel (10,000#/□″) is comparatively high.

Illustrative Prob. 114c. Calculate the average intensity of vertical shear* for the typical joist discussed in Illustrative Prob. 114*b*.

$$V = \frac{w \cdot L}{2} = \frac{128 \times 14.83}{2} = 950\#$$

$$v = \frac{V}{A_w} = \frac{V}{d \cdot t} = \frac{950}{7 \times 0.144} = 943\#/\square''$$

v (allowable) = 10,000#/□″ *Shear O.K.*

* This value is representative of the maximum value of horizontal shear, and if it is sufficiently below the allowable, the horizontal shear may be assumed safe.

The **deflection** of the joists is an important part of the investigation, however, as in the majority of cases the plastered ceiling is directly attached to the soffits of the joists, and furthermore, the flooring is applied directly to the tops. When deflection controls this feature, it is usually indicated in tables. (See footnote, Table 41.)

Illustrative Prob. 114d. Show that the deflection of the joist selected in Illustrative Prob. 114*b* is within safe limits.

$$I = 11.2''^4 \text{ (Table 39)} \qquad 14'\text{-}10'' = 178'' = l$$

$$D = \frac{5\,W \cdot l^3}{384\,E \cdot I} = \frac{5 \times (128 \times 14.83) \times (178)^3}{384 \times 30{,}000{,}000 \times 11.2} = 0.414''$$

$$D \text{ (allowable)} = \frac{l}{360} = \frac{178}{360} = 0.493''$$

Deflection O.K.

The methods illustrated in the above problems are not usually employed to design steel joist floors. A study of the conditions will readily show that safe-load tables may be easily and safely devised for this purpose. The following reasons give evidence of the wisdom of such a suggestion:

(1) The number of sections which may be used is small,

(2) the unit live loads are usually small, and vary within narrow limits (40# to 100#),

(3) the joists carry uniform loads in the majority of cases,

(4) the spans are unusually well standardized, due to the room sizes ordinarily encountered in the types of buildings for which this construction is especially suitable, and

(5) that under the usual circumstances, the questions of shear and buckling are not controlling elements in the design.

The above situations do not hold as commonly for rolled sections and for reinforced concrete. Table 41 gives the total loads in #/□' of floor surface that various combinations of Berloy joists can safely carry. These values are based upon a moment of $\frac{W \cdot L}{8}$. When the joists run continuous over a supporting beam at one end, a coefficient of $\frac{W \cdot L}{9}$ may be used, and when continuous at both supports, $\frac{W \cdot L}{10}$ is employed. A designer should select the most **economical arrangement** of joists that the conditions will allow. Table 41 is a distinct advantage in such a case.

Illustrative Prob. 114e. Select an economical arrangement of joists for a L.L. of 100#/□', a span of 16'-0", and wood finish flooring.

$$\begin{aligned} \text{L.L.} &= 100 \\ \tfrac{7}{8}'' \text{ Fin. Flr.} &= 3 \\ \tfrac{7}{8}'' \text{ Sub Flr.} &= 3 \\ 2'' \text{ Concrete} &= 25 \\ \text{Joists} &= 4 \text{ (Table 40)} \\ \text{Ceiling} &= \underline{10^*} \\ \text{T.L.} &= 145\#/\square' \end{aligned}$$

Referring to Table 41, the following combinations are

11"-9.5# joists 24" o.c. — $9.5 \div \frac{24}{12} = 4.76\#/\square'$
10"-8.0# joists 19" o.c. — $8.0 \div \frac{19}{12} = 4.42\#/\square'$
9"-7.0# joists 16" o.c. — $7.0 \div \frac{16}{12} = 5.23\#/\square'$
8"-6.1# joists 12" o.c. — $6.1 \div \frac{12}{12} = 6.1\ \#/\square'$

From the above it is seen that 10" joists, 19" o.c., are most economical as far as weight is concerned. In particular cases, other factors may influence the selection, such as the thickness of floor construction and consequent addition to columns and walls, the number of joists to place, headroom, and so on.

The table cannot be used when special instances of loading are encountered. When **non-bearing partitions** occur in a direction **perpendicular to that of the joists,** as in Fig. 170, the joists must sustain a concentrated load in addition to the uniform load. The value of the former may be expressed as

$$P = w_p \cdot s \cdot h$$

in which

P = the concentrated load in lbs.,
s = the spacing of joists in feet,
h = the height of the partition in feet, and
w_p = the weight of the partition in lbs. per superficial sq. ft.

Illustrative Prob. 114f. What size of joists is requires for the data of Illustrative Prob. 114*e* if a partition crossed them 5'-0" from one end? Story height 11'-0".

$$\text{Uniform load} = 145 \times \tfrac{19}{12} = 230\#/\text{ft. (Prob. 114}e)$$

4" channel stud partition, three coats of plaster each side $= 110\#/\text{sq. yd.} = 12\#/\square'$†

$$\text{Partition height} = 11'\text{-}0'' - (2 + 2 + 10) = 9.83'$$

$$P = w_p \cdot s \cdot h = 12 \times \tfrac{19}{12} \times 9.83 = 187\#$$

$$R_2 = 187 \times \tfrac{11}{16} + \frac{230 \times 16}{2} = 1970\#$$

$$V_0,\ 1970 - 5 \times 230 - 187 = 630$$
$$630 \div 230 = 2.74'$$
$$2.74 + 5 = 7.74' \text{ from } R_2$$

$$M_{max} = 1970 \times 7.74 - \frac{230 \times (7.74)^2}{2} - 187 \times 2.74$$
$$= 7850'\# = 94{,}100''\#$$

$$M = \frac{s \cdot I}{c} = 94{,}100 = 16{,}000\frac{I}{c} \qquad \frac{I}{c} = 5.88''^3$$

Referring to Table 41, 10" Joists, 19" o.c. may still be used.

* Some architects specify that the metal lath attached to the soffits of the joists shall be back plastered in order to secure additional fire protection. In such cases, the weight should be included in the load per sq. ft.

† For weights of partitions, see Index.

TABLE 41

TOTAL SAFE UNIFORM LOADS ON STANDARD BERLOY JOISTS IN #/□' OF FLOOR AREA

Joists Spaced 24" C.C.

Depth of Joist	4"	5"	6"	7"	8"	9"	10"	11"	12"
Weight per Linear Ft.	3.7	4.2	4.7	5.5	6.1	7.0	8.0	9.5	10.5
Clear Span in Feet									
6	193								
7	142	191							
8	108	146	192						
9	77	115	152	211					
10	56	94	123	171					
11	42	70	102	141					
12	32	54	85	119	156				
13		43	67	101	133	167			
14		34	54	87	115	144			
15			44	71	100	126	158	199	237
16			37	59	88	111	139	175	209
17				49	74	98	123	155	185
18				41	62	88	110	139	165
19					53	75	99	124	148
20					45	64	89	112	134
21					39	56	78	102	121
22						48	67	93	110
23						42	59	82	101
24							52	72	93
25							46	64	83
26							41	57	74

Joists Spaced 19" C.C.

Depth of Joist	4"	5"	6"	7"	8"	9"	10"	11"	12"
Weight per Linear Ft.	3.7	4.2	4.7	5.5	6.1	7.0	8.0	9.5	10.5
Clear Span in Feet									
6	244								
7	179	240							
8	137	184	241						
9	97	146	191	266					
10	71	118	155	216					
11	53	89	128	178					
12	40	69	107	149	196				
13		54	85	127	167	211			
14		43	68	110	145	182			
15			55	90	126	159	199	252	290
16			46	74	110	140	175	221	263
17				62	93	124	155	196	233
18				52	78	110	138	175	208
19					66	95	124	157	186
20					57	81	112	141	169
21					49	70	98	128	153
22						61	85	117	139
23						53	74	103	127
24							65	91	117
25							57	80	104
26							51	71	93

Joists Spaced 16" C.C.

Depth of Joist	4"	5"	6"	7"	8"	9"	10"	11"	12"
Weight per Linear Ft.	3.7	4.2	4.7	5.5	6.1	7.0	8.0	9.5	10.5
Clear Span in Feet									
6	289								
7	212	286							
8	163	219	287						
9	116	173	227	316					
10	84	140	184	256					
11	63	106	152	212					
12	48	82	128	178	234				
13		64	101	152	199	251			
14		51	81	130	172	216			
15			66	107	150	188	236	299	356
16			54	88	131	166	208	262	313
17				74	110	147	184	232	277
18				62	93	131	164	208	247
19					79	113	147	186	221
20					68	96	133	168	200
21					58	83	116	152	182
22						72	101	139	165
23						63	88	123	151
24							77	108	139
25							68	95	124
26							61	85	110

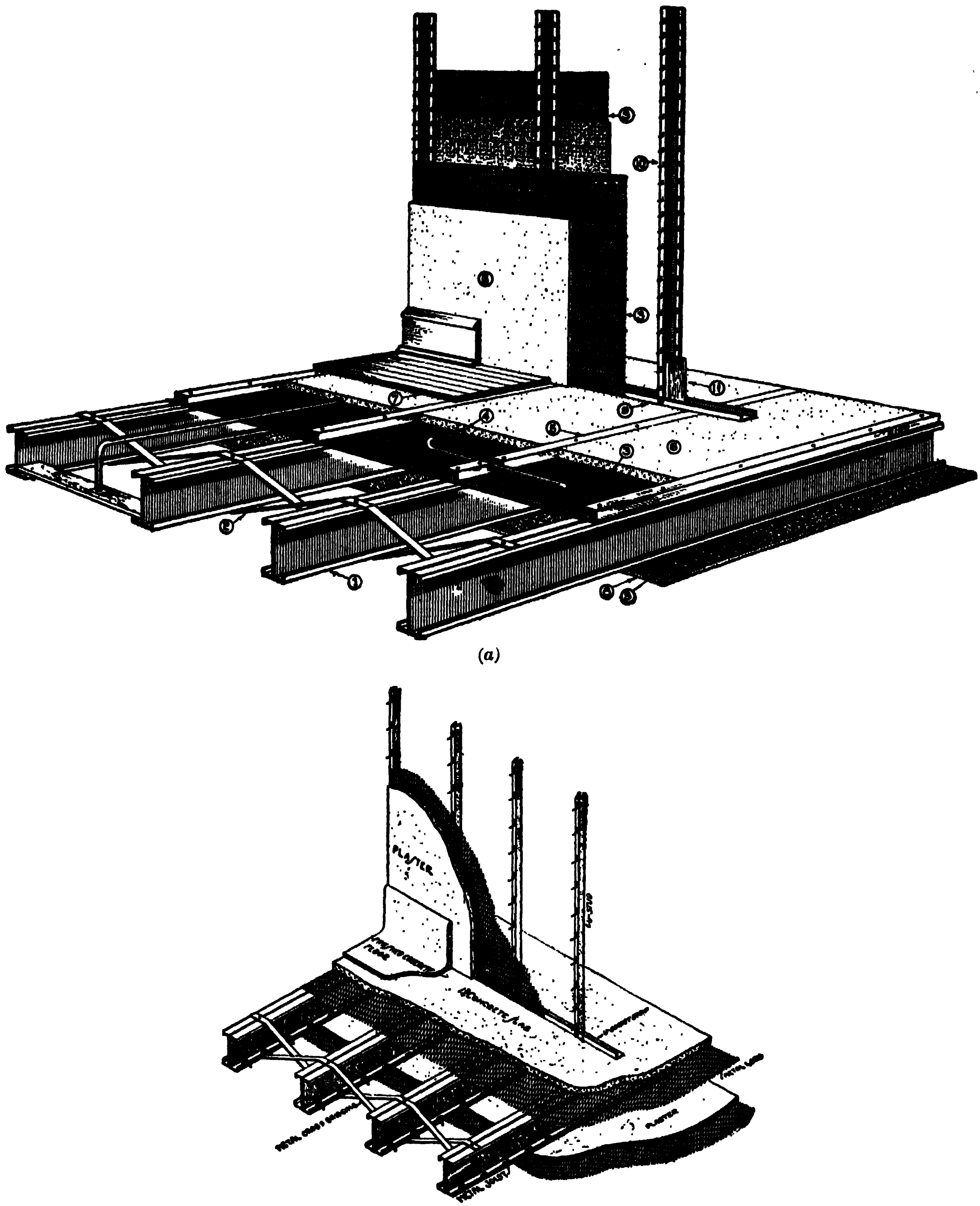
Joists Spaced 12" C.C.

Depth of Joist	4"	5"	6"	7"	8"	9"	10"	11"	12"
Weight per Linear Ft.	3.7	4.2	4.7	5.5	6.1	7.0	8.0	9.5	10.5
Clear Span in Feet									
6	385								
7	283	381							
8	217	292	383						
9	154	231	303	421					
10	112	187	245	341					
11	84	141	203	282					
12	64	109	170	237	311				
13		86	134	202	265	334			
14		68	108	174	229	288			
15			88	142	199	251	315	398	474
16			73	118	175	221	277	350	417
17				98	147	196	246	310	369
18				82	124	175	219	277	329
19					105	150	197	248	295
20					90	128	177	224	267
21					78	111	155	203	242
22						96	134	185	220
23						84	117	164	202
24							103	144	185
25							91	127	165
26							81	113	147

Based upon a moment of $\frac{W \cdot L}{8}$ and a maximum fibre stress of 16,000#/□".

Deflection does not exceed $\frac{1}{360}$th of the span in inches. It should be remembered that the values tabulated are total loads and include the dead weight.

The values are based upon the contingency that the joists are properly braced with bridging, lath, etc.



(a)

(b)

FIG. 170. BERGER FLOOR AND PARTITION CONSTRUCTION

(a) I-joists and channel studs

(b) I-joists and U-studs

1. I-joists
2. bridging
3. metal lath
4. electric conduits
5. nailing strips
6. concrete fill
7. finished wood floor
8. plaster
9. socket strip
10. studs
11. nailing block

Instead of carrying computations to the degree of accuracy illustrated above, an approximate solution is generally used, namely, that of assuming the span to be 2'-0" greater than the actual, and using Table 41.

Illustrative Prob. 114g. Check Illustrative Prob. 114*f* by the approximate method.

T.L. = 145#/□' Actual span = 16'-0"
Assume span 18'-0".
From Table 41, w_0 = 138#/□' O.K. practically
Use 10" Joists — 19" o.c.

When a **bearing partition** crosses the joists **at right angles,** as is the case when a change of framing supports occurs, the amount of the concentration usually requires the use of heavier members. When a **non-bearing partition** runs **parallel** with the joists, the common practice is to use double, metal joists in the same manner as for wood joist construction. This is practically always more than sufficient for strength, but it is required to provide a support for the channel track of the partition, as illustrated in Fig. 170.

Illustrative Prob. 114h. If the partition in Illustrative Prob. 114*f* were parallel to the joists, would a double joist be sufficient to carry the load?

Load per foot from floor = 230# (Prob. 114*f*)
Load per foot from partition = 12 × 9.83 = 118#
Total load per foot = 348#

$$M = 1.5\,w \cdot L^2 = 1.5 \times 348 \times (16)^2 = 134{,}000''\#$$

$$\frac{I}{c} = \frac{134{,}000}{16{,}000} = 8.35''^3$$

2-10" Joists = 2 × 6.65 = 13.30"³ (Table 39)

Thus a double joist is satisfactory as is the usual case, and the above computations are not generally made. The live load need not have been considered on the space the partition occupied.

If a **bearing partition** occurs in a direction **parallel** to the joists, it is usually necessary to introduce a structural steel beam to carry the load. If possible, bearing partitions should be built continuous with the one occurring in the story below.

The previous discussion has all been based upon the fact that the joists are **laterally supported.** Such support is obtained **by** the use of **bridging,** in addition to that provided by the metal lath and slabs. The typical bridging is shown in Fig. 169, and usually consists of #20 gauge galvanized steel 1" wide, drawn taut in alternate diagonals and fastened between the webs with 6d nails. In addition to the lateral support afforded by such fastenings, the joists are also held in place vertically during erection. The bridging also helps to transfer any concentrated load to other joists. Lateral supports of this nature are more important in metal lumber floors than in wood joist floors, and more rigid requirements are necessary. The spacing should not exceed 6'-0", and for joist spans greater than 12'-0", the bridging should be placed at least at the third-points of the span.

Illustrative Prob. 114i. Determine the sizes of joists and I-beams required for a panel 20'-0" × 20'-0". L.L. 60#/□', double floor and plastered ceiling.

L.L.	=	60
Fin. Flr.	=	3
Sub Flr.	=	3
2" Concrete	=	25
Joists	=	4
Ceiling	=	10
T.L.	=	105#/□'

From Table 79,
11"-9.5# Joists 24" o.c. = 4.75#/□'
10"-8.0# Joists 19" o.c. = 5.05#/□"
10"-8.0# Joists 16" o.c. (No advantage)
9"-7.0# Joists 12" o.c. = 7.0#/□'
Use 11"-9.5# Joists, 24" o.c.

Load on beam.

105 × 20 = 2100#/ft.
Bm. = 55
F.P. = 35
2190

$$M = 1.5\,w \cdot L^2 = 1.5 \times 2190 \times (20)^2 = 1{,}313{,}000''\#$$

$$\frac{I}{c} = \frac{1{,}313{,}000}{16{,}000} = 82.1''^3.$$

Use 18 I 54.7
(Refer to Table 33)
Use 8 *I* 18 *Column Ties*
Use 3 × 2½ × ¼ *Shelf Angles*

Figure 171 shows a typical engineer's sketch.

Prob. 114j. Check the other sizes of the joists shown on Pl. 12. Typical floor. Refer to Illustrative Prob. 114*b*.

Prob. 114k. Select an economical arrangement of joists for a L.L. of 60#/□' and a span of 14'-0". Typical floor.

Prob. 114l. What size of joists is required in Prob. 114*k* if a partition (12# per superficial □' and 10'-0" high) crosses them at a distance of 6'-0" from one end? Check the result by the approximate solution.

Prob. 114m. Provide an arrangement if the joists of Probs. 114*k* and 114*l* are to carry the partition in a direction parallel with their length.

115. Framing Around Openings.

At points where openings occur, special construction is necessary. Pressed steel sections should only be used to frame around **small openings** such as those for vents, flues, small skylights, chimneys, and so on. Special heavy #12 gauge pressed steel joists are used for the trimmers and headers in such work. Flange connections are made as illustrated in Fig. 172, or standard #11 gauge angle connections are used, particularly when partitions occur around the openings. For very small openings the standard sections are often used. When **large openings** occur, such as at stairways, elevators and large skylights, the principal framing should be made with structural steel shapes, as illustrated in Fig. 173.

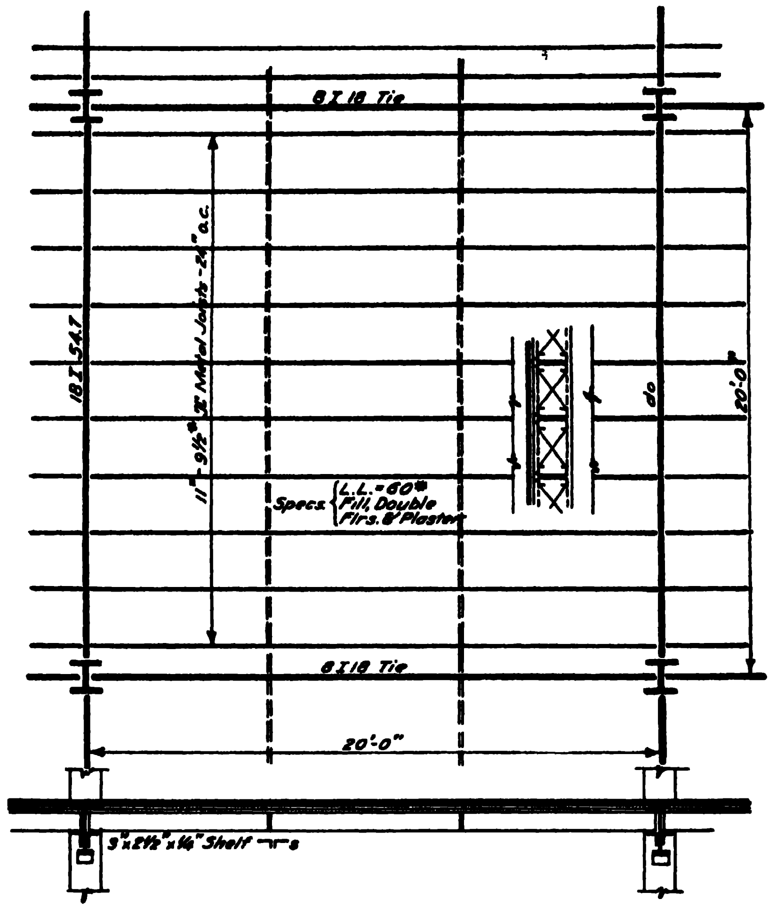


FIG. 171. TYPICAL STEEL JOIST FRAMING PLAN

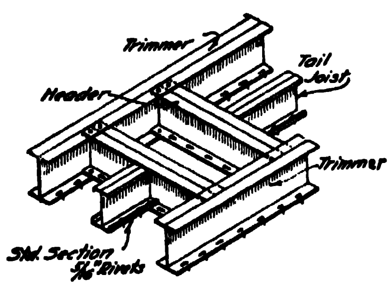


FIG. 172

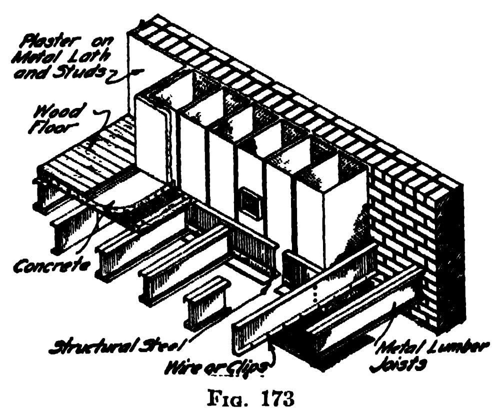


FIG. 173

116. Determination of Rolled Steel Girder Sizes.

Again, the calculations for the sizes of the beams required to support the joists are usually based upon a uniform distribution of load. The live load may be reduced if the tributary area of floor is large enough (Art. 93).

Illustrative Prob. 116a. Check the size of steel beam " A " shown on Pl. 12.

$$L = \frac{(33'\text{-}0'') - 2(1'\text{-}8'')}{2} = 14'\text{-}10'' \text{ inside face of wall to}$$
center-line of column.

Assume 6" bearing.

L (c.c. bearings) = 14'-10" + 3" = 15'-1"

L.L. = 50#/□' D.L. = 40#/□' T.L. = 90#/□'

$w = 90 \times 12.17 = 1097$#/ft.

Beam wt. = 25#

F.P. = 25

50#/ft. $\qquad w$ (total) = 1097 + 50 = 1147#/ft

$M = 1.5\, W \cdot L = 1.5 \times 1147 \times 14.83 \times 15.1 = 385{,}000''$#

$$\frac{I}{c} = \frac{385{,}000}{16{,}000} = 24.0''^3$$ Use 10 I 25.3 (Table 33)

$V = \frac{1147 \times 14.83}{2} = 8500\#$

$v = \frac{8500}{10 \times 0.31} = 2740\#/\square''$ Shear O.K.

$L = 2\,d = 2 \times 10 = 20'\text{-}0''$ Deflection O.K.

$\frac{8500}{175} = 48.6\square''$. Use std. beam seat.

Use $8 \times \frac{3}{8} \times 0'\text{-}8''$ bearing plate.

In order to provide proper **fire protection,** the steel beams should be encased with fire resisting materials. Figure 174 (*a*) shows one form of such construction which is effective and economical. Another method is shown in (*b*). This is more effective than (*a*) but it is not considered as necessary by many contractors, and it adds weight to

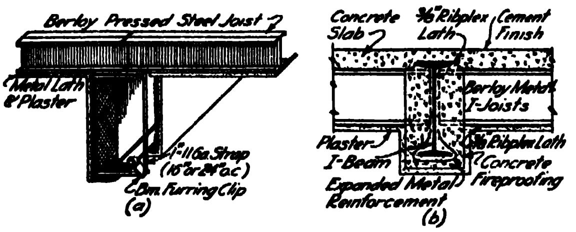


FIG. 174

that which the beam must carry, as well as being more expensive to form. In either instance, an **allowance** should be made **for the beam haunch in** the **calculations,** as illustrated in the problem above. For (*a*) an allowance of $8\#/\square'$ of fireproofed surface is sufficient, and in (*b*) the usual weight of $150\#$/cu. ft. of concrete is used. When structural steel floor ties are required for the columns (they should be braced in at least two directions), a tie beam may be made arbitrarily 6″ or 8″ and thus concealed within the floor depth.

Prob. 116b. Check the sizes of the rolled steel shapes shown on Pl. 12. L.L. $= 50\#/\square'$, typical floor.

Prob. 116c. Design a typical interior panel 18′-0″ × 18′-0″ (joists and supporting beams) for a L.L. of $100\#/\square'$. 1″ granolithic finish floor, no fill.

117. Details at End Supports of Joists.

The details at the end supports of metal joists are similar to those for wood joists in many ways. A sufficient bearing length is required, as for all beams, although the additional precaution of simple fire protection is necessary. It is also important to make certain that there is no danger of the webs crippling at the supports (Art. 13). The allowable stress in such an instance may be determined from the formula previously discussed, namely,

$$f_b = 16{,}000 - 120\frac{d}{t}.$$

The allowable end reaction, as before, is expressed by

$$R = f_b \cdot t\left(a + \frac{d}{4}\right). \qquad \text{(Art. 13.)}$$

Illustrative Prob. 117a. Is the joist of Prob. 114*f* safe for buckling if a $2\frac{1}{2}''$ bearing is used?

$R_1 = 1970\#$ 10″ Joist, $t = 0.156$

$f_b = 16{,}000 - \frac{120 \times 10}{0.156} = 8200\#/\square''$

$R = 8200 \times 0.156\left(2.5 + \frac{10}{4}\right) = 6400\#$ (allowable).

Joist O.K. in buckling.

From the above illustration, it is seen that joists are practically always safe in buckling, if a sufficient length of bearing is maintained (as the conditions assumed are extreme), and for all ordinary circumstances, the **buckling investigation may be omitted** at the end supports, as well as for any occasional concentrations (Art. 13).

When considerations of headroom are not important criteria, or the supporting beam may be concealed in a partition, or when the soffits of the beams may be allowed to cause ledges at the sides of the room or panels in the ceiling of the room, the joists may be allowed to run **over the tops of** the **supporting girders,** as illustrated in Fig. 175. The joists should not be allowed to butt, as in (*a*), unless the breadth of the beam flange is $5\frac{1}{2}''$ or larger.

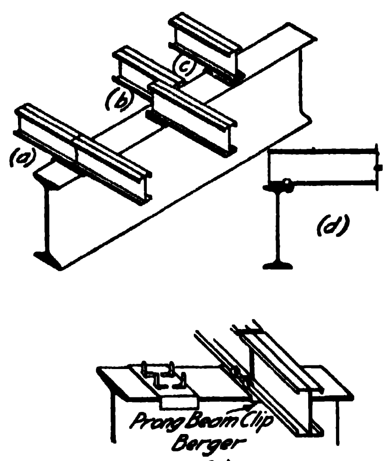


FIG. 175

This is specified because a minimum bearing length of $2\frac{1}{2}''$ is essential, and there should be a clearance of $\frac{1}{2}''$ between the ends of the joists. A $2\frac{1}{2}''$ distance is not theoretically required for direct bearing, as the allowable bearing stress of steel upon steel is relatively very high ($20{,}000\#/\square''$), but it is necessary in order to provide stiffness at the support and to prevent the edges of the flanges from bending and the

web from buckling. When the joists lap, as in (*b*), or single joists occur, as in (*c*), the ends should extend slightly beyond the center-line of the beam. In such cases, special beam clips are used to hold the joists laterally, as shown in (*e*), although some contractors do not use any, and erroneously claim that the bridging and concrete are sufficient to provide this requirement. Other builders use rivets or special mortise and tenon angle connections. In any case, some sort of fastening at the point of bearing should always be provided. Figure 176 shows alternate details.

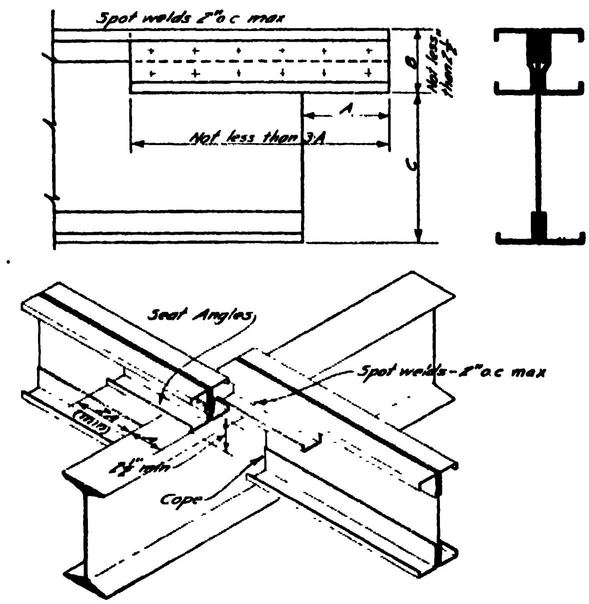


Fig. 176

Usually headroom is an important factor, however, and the joists are often **supported by shelf angles** attached to the supporting beam, as illustrated in Fig. 177 (*a*). A minimum 3" × 2½" × ¼" angle should be used to provide the necessary 2½" bearing length. Figure (*a*) is used when the joists are of the same depth, while (*b*) and (*c*) are used when the joists are of different depths, (*b*) showing the beveled and square-cornered copings, respectively, and (*c*) the advisable method of eliminating coping. Figure (*d*) shows the method of attachment to channels surrounding large openings. When shelf angles are used, no attachment is considered necessary, as the bridging and concrete supply sufficient stiffness. An alternate detail is shown in (*e*), and another in (*f*) which employs small strap hangers, although these two details are seldom used as they are expensive and not very satisfactory.

When the joists are **supported by** steel stud bearing **partitions** or walls, the detail in Fig. 178 is commonly used. The bearing length is made the full width of the channel track, and the joists are attached with $\frac{5}{16}$" rivets or bolts. When the joists **bear upon masonry walls,** the length of the bearing should be made equal to one-half the depth of the joist, or 4" as a minimum. No bearing plates are required as the area provided is ample if such lengths are used.

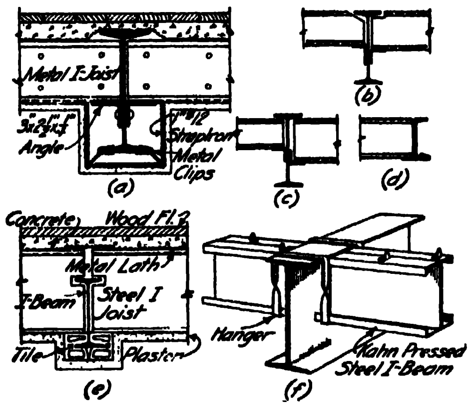


Fig. 177

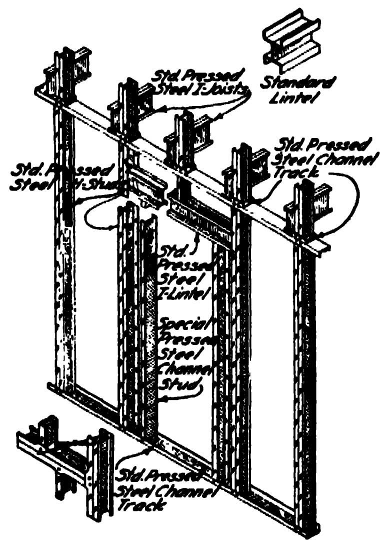


Fig. 178

Illustrative Prob. 117b. What length of bearing should be used in Prob. 117*a* if the joists are to rest upon a 12" brick wall (p = 175#/□")?

$$\frac{d}{2} = \frac{10}{2} = 5'' \qquad \frac{1970}{175} = 11.2□''$$

R_1 = 1970#

Width of flange of 10" joist = 4½"

$$\text{Theoretical length of bearing} = \frac{11.2}{4.5} = 2.5''.$$

Use 6" for practical reasons.

It is seen that the rules of establishing bearing lengths will provide ample areas. The data used are for a case of extreme loading.

A lateral tie is often provided by using a strip of the typical bridging, running over the tops of the joists near their ends, as shown in Fig. 179 (*a*). **End anchors** about 6'-0" o.c. may be used as illustrated in Fig. 179 (*c*) and (*d*), although many contractors correctly contend that the shape of the joist is sufficient anchorage in itself. When a joist runs parallel to a wall, **side anchors** may be used, as shown in (*e*), using a spacing of about 6'-0" o.c., and carrying them across at least one interior joist.

Prob. 117c. Write a specification to cover the details of metal lumber I joist supports and anchorage.

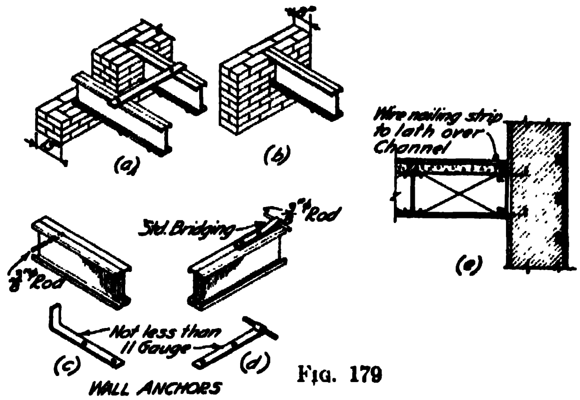


FIG. 179

SECTION 8B

BAR JOISTS*

118. The Trussed Type.

Another form of metal joist which has been recently placed upon the market is the Massillon Bar Joist.† Figure 180 shows a typical unit. It consists of 5 structural grade steel bars (minimum $\frac{3}{8}''\phi$), shop fabricated and electric-arc welded into the shape of a Warren truss with cantilever ends.

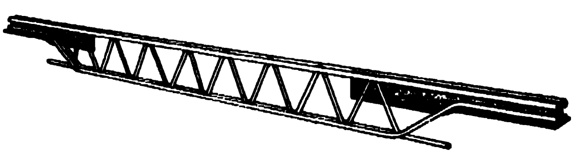


FIG. 180

These are manufactured in 22 standard units, with 5 depths, namely, 8", 10", 12", 14" and 16", and the joists of the same depth are made in various standard lengths. Except for the 8" depth, there are three standard lengths. The 8" joist is made in six lengths and there are also four special 8" sections, adaptable for residence work. All the joists are marked a number such as 8, 10, and so on, followed by a letter, A, B, etc. The first refers to joist depth and the second to length. If a joist must be cut for some special reason, it is marked "12-B cut 3", for instance. The residence joists are marked specially, such as 8 R-A, and the like. By this series, spans from 4'-0" to 30'-6" may be suited. Two standard header sections are used for framing around small openings. For large openings, the regular carrying frame should be used. The ends of the bars are connected to a plate which acts as a gusset plate, bracing plate, and bearing plate. Each joist is $2\frac{1}{2}''$ deep at each end so that it conforms to brick coursing and supplies sufficient strength. The center of gravity of the section is below the points of support so that the joist is always stable.

Figure 181 shows a typical view of the floor construction. The finish floor may be any desired. If wood is used, nailing strips are provided, usually at right angles to the joists and placed 16" o.c. The carrying floor is a 2" concrete slab, which is formed by flat diamond mesh expanded metal, weighing 4.0# per sq. yd. The joists are spaced from 12" to 24" o.c., depending upon the loading. The ceiling is formed by wiring pencil rods or pencil channels to the bottoms of the joists, and attaching 3.4# flat diamond mesh lath to them for a plaster base, or $3\frac{1}{2}$# ribbed ceiling lath may be used instead. The joists are supported on the tops of the I beams or by walls or partitions, and they may be lapped by when necessary to avoid cutting. Although not theoretically required, the joists should be braced laterally. This is done by using standard bridging wire in the planes of the tops and bottoms of the joists. The wire should be placed at mid-span up to 16'-0" lengths, at the third-points for 16'-0" to 24'-0" lengths, and at the quarter-points of the span for 24'-0" to 30'-0" lengths. Figure 182 shows a typical installation.

A distinct advantage of this type of construction is the possibility of obtaining simple and efficient piping layouts for the plumbing, heating, ventilation, and wiring installations, as the pipes and conduits can be run in almost any direction. Since the joists rest on top of the I beams, room is left for the piping

* In addition to the "Massillon" and Rivet-grip types of bar joists, there are a number of others of similar nature. One type is that manufactured by the Bates Expanded Steel Truss Co., East Chicago, Indiana. This joist does not rely upon rivets, bolts or welds in shear or tension, but is expanded from previously slitted and heated shapes. These are available in depths from 8" to 18", varying by inches, and in any desired length up to 35'-0". Each joist has an 8" variable length. Another type is manufactured by the Havemeyer Bar Co.

† Patented and manufactured by the Massillon Steel Joist Company, Massillon, Ohio.

to run over the latter. Future changes and repairs necessitate only removing the ceiling, without cutting up the floor. Other advantages are the quick erection and relatively light weights of the floor, the latter averaging from 35# to 40#/□′. In a fire-resisting way, the joists are superior to some other types, because the metal is of larger section and not thin, the thinnest metal being $\frac{1}{4}''$ in the bearing plates and all the other is $\frac{3}{8}''$ in diameter or more. The open air space is an advantage in diffusing heat rather than localizing it.

Table 42 gives total safe loads in #/□′, uniformly distributed, based upon the American Institute of Steel Construction 1923 Specifications, allowing a maximum tensile stress of 18,000#/□″ (see Art. 9). For a stress of 16,000#/□″, the loads may be reduced in direct proportion. All loads are computed for the net span. The width of the beam flange should be deducted to obtain the latter. The live load is determined by deducting the weight of the floor construction, which is about 40#/□′ for the usual construction. The table is based upon the assumption that the joists are braced laterally by the concrete or wood flooring. Each selection should be made upon a basis of economy in weight, with the depth, as affecting headroom, being considered also. The joists are spaced to develop their capacity.

Illustrative Prob. 118a. Select a size of joist to span 16′-0″ c.c. bearings and to carry a live load of 40#/□′. Double wood finish floor.

L.L. = 40		
1″ Fin. Flr. = 3		Span (net) = 15′-6″.
1″ Sub. Flr. = 3		From tables,
2″ Concrete = 25		8 R-B, 15″ o.c., or
Ceiling = 10		8 R-A, 16″ o.c., or
Joists = 4		10-B, 30″ o.c.
T.L. = 85#/□′		

The 10″ joists should not be spaced farther than 24″ apart so that they are not considered and are intended for heavier live loads.

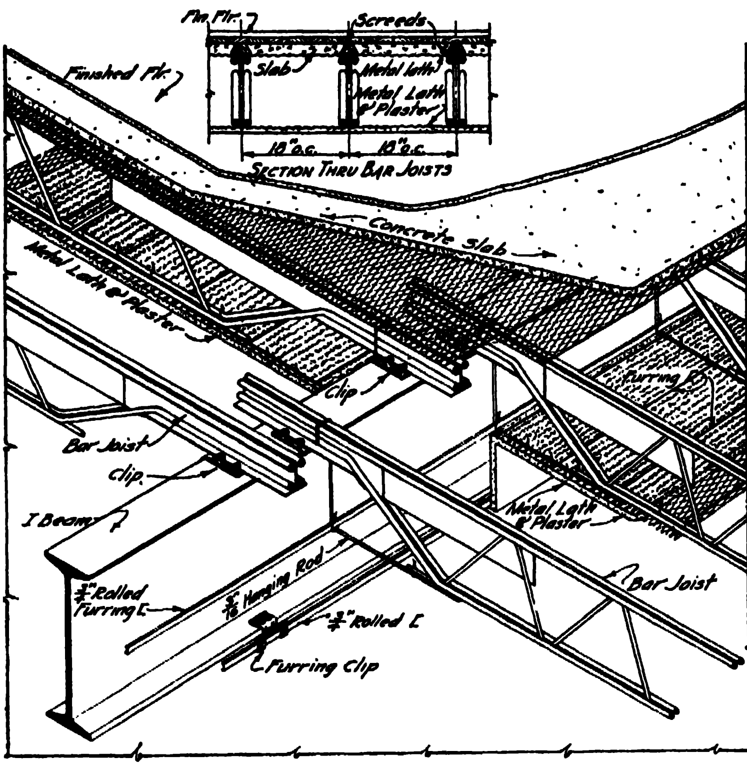


Fig. 181

8 R-B, wt. = 58#	58 ÷ 15 = 3.97#
8 R-A, wt. = 64#	64 ÷ 16 = 4.00#
	Use 8 R-B Joists.

Prob. 118b. Select a size of joist to span 18′-0″ c.c. of bearings and to carry a live load of 60#/□′. Double wood finish floor.

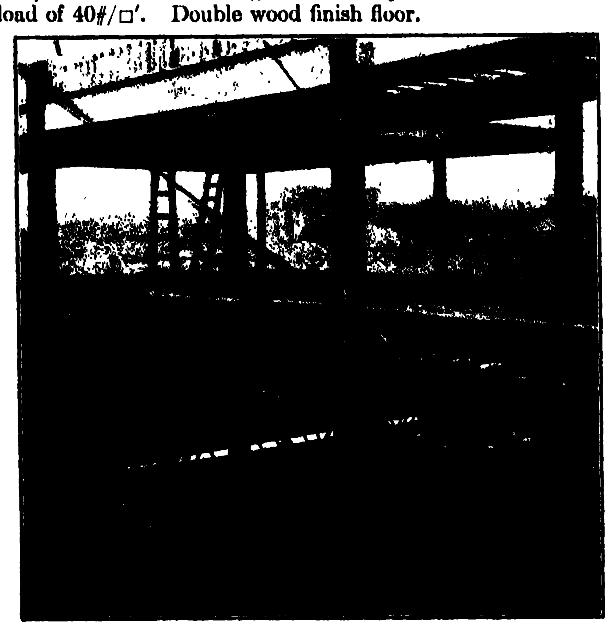


Fig. 182

119. Another Type of Bar Joist.

Another form of the metal joist is the rivet-grip Steel Joist.* It is quite similar to the Massillon joist, having its own particular features and specifications. The joist consists of special deformed bars fabricated in the form of a truss, as illustrated in Fig. 183(a). These units are detailed and made specially for each job and are shipped ready to be placed. The lightest web members are $\frac{1}{4}'' \times \frac{9}{16}''$, thus eliminating thin metal which would be impaired by corrosion. All joists are shop painted to prevent

* A patented form manufactured by Rivet-Grip Steel Company, Cleveland, Ohio, formerly the Concrete Reinforcing and Engineering Company.

TABLE 42

TOTAL SAFE LOADS FOR MASSILLON BAR JOISTS IN #/□'

Maximum tensile stress = 18,000 #/□"*

Span	Joist	Weight per Piece	Total Load	Spacing C-C of Joists in Inches															
				12	14	15	16	17	18	19	20	21	22	23	24	25	26	28	30
4'-0"	8-F	19	2940	735	630	589	552	520	490	465	441	420	401	384	368	353	340	316	294
4'-6"	"	"	2502	556	477	445	418	393	372	352	334	318	304	290	278	268	257	239	223
5'-0"	"	"	2190	438	376	350	328	310	292	277	263	251	239	229	219	210	202	188	175
5'-6"	"	"	1947	354	304	284	266	250	236	224	212	202	193	185	177	170	163	152	142
6'-0"	"	"	1758	293	251	234	220	207	195	185	176	168	160	153	147	141	135	126	117
	8-E	23	2214	369	316	295	277	260	246	233	222	211	201	193	184	177	170	158	148
6'-6"	"	"	2112	325	278	260	244	230	217	205	195	186	177	170	163	156	150	139	130
7'-0"	"	"	1981	283	242	226	212	200	189	179	170	162	154	148	142	136	131	121	113
7'-6"	"	"	1792	239	205	191	179	169	159	151	143	136	130	125	120	115	110	102	96
8'-0"	"	"	1560	195	167	156	146	138	130	123	117	111	106	102	98	94	90	84	78
	8-D	30	2512	314	269	251	236	222	210	198	188	179	171	164	157	151	145	135	126
8'-6"	"	"	2320	273	234	218	205	193	182	173	164	156	149	142	136	131	126	117	109
9'-0"	"	"	2169	241	207	193	181	170	161	152	145	138	132	126	121	116	111	103	96
9'-6"	"	"	2014	212	182	170	159	150	141	134	127	121	116	111	106	102	98	91	85
	8-C	40	2992	315	270	252	236	222	210	199	189	180	172	164	157	151	145	135	126
10'-0"	8-D	30	1890	189	162	151	142	133	126	119	113	108	103	99	95	91	87	81	76
	8-C	40	2820	282	242	226	212	199	188	178	169	161	154	147	141	135	130	121	113
10'-6"	"	"	2667	254	218	203	190	179	169	160	152	145	138	132	127	122	117	109	101
11'-0"	"	"	2453	223	191	178	167	157	149	141	134	127	122	116	112	107	103	95	89
	8 R-D	"	1815	165	142	132	124	117	110	104	99	94	90	86	83	79	76	71	66
11'-6"	8-C	"	2311	201	172	161	151	142	134	127	121	115	110	105	101	97	93	86	80
	8 R-D	"	1690	147	126	118	110	104	98	93	88	84	80	77	74	71	68	63	59
	8-B	52	3174	276	236	221	207	195	184	174	166	158	151	144	138	133	128	118	111
12'-0"	8 R-D	40	1608	134	115	107	101	95	89	84	80	77	73	70	67	64	62	57	54
	8-B	52	2976	248	212	199	186	175	165	157	149	142	135	130	124	119	115	106	99
12'-6"	8 R-D	40	1525	122	105	98	92	86	81	77	73	70	67	64	61	59	56	52	49
	8-B	52	2800	224	192	179	168	158	149	141	134	128	122	117	112	107	103	96	90
	8 R-C	45	1612	129	111	103	97	91	86	81	77	74	70	67	64	62	59	55	52
13'-0"	8-B	52	2652	204	175	163	153	144	136	129	122	117	111	107	102	98	94	87	82
	8 R-C	45	1521	117	100	94	88	83	78	74	70	67	64	61	58	56	54	50	47
	8-A	70	3406	262	224	210	197	185	175	166	157	150	143	137	131	126	121	112	105
13'-6"	8 R-C	45	1444	107	92	86	80	75	71	67	64	61	58	56	53	51	49	46	43
	8-A	70	3254	241	206	193	181	170	161	152	145	138	132	126	121	116	111	103	96
	10-C	75	4266	316	271	253	237	223	211	200	190	181	172	165	158	152	146	135	126
14'-0"	8 R-C	45	1372	98	84	78	73	69	65	62	59	56	53	51	49	47	45	42	39
	8-A	70	3052	218	187	175	164	154	145	138	131	125	119	114	109	105	101	94	87
	8 R-B	58	1932	138	118	110	103	97	92	87	83	79	75	72	69	66	64	59	55
	10-C	75	4060	290	248	232	218	205	193	183	174	166	158	151	145	139	134	124	116
14'-6"	8-A	70	2900	200	172	160	150	141	133	126	120	114	109	104	100	96	92	86	80
	8 R-B	58	1827	126	108	101	94	89	84	79	75	72	69	66	63	60	58	54	50
	10-C	75	3842	265	227	212	199	187	177	167	159	151	145	138	133	127	122	114	106

* When a stress of 16,000 #/□" is specified, values may be obtained by direct proportion.

TABLE 42 — *Continued*

Span	Joist	Weight per Piece	Total Load	Spacing C-C of Joists in Inches 12	14	15	16	17	18	19	20	21	22	23	24	25	26	28	30
15′-0″	8 R-B	58	1755	117	100	94	88	83	78	74	70	67	64	61	58	56	54	50	47
	10-C	75	3630	242	208	194	182	171	161	153	145	138	132	126	121	116	112	104	97
	10-B	85	3930	262	224	210	197	185	175	165	157	150	143	137	131	126	121	112	105
15′-6″	8 R-B	58	1690	109	93	87	82	77	73	69	65	62	59	57	54	52	50	47	44
	8 R-A	64	1751	113	97	90	85	80	75	71	68	65	62	59	57	54	52	48	45
	10-B	85	3766	243	208	194	182	172	162	153	146	139	133	127	122	117	112	104	97
16′-0″	8 R-A	64	1664	104	89	83	78	73	69	66	62	59	57	54	52	50	48	45	42
	10-B	85	3616	226	194	181	170	160	151	143	136	129	123	118	113	108	104	97	40
16′-6″	8 R-A	64	1584	96	82	77	72	68	64	61	58	55	52	50	48	46	44	41	38
	10-B	85	3399	206	177	165	155	146	137	130	124	118	112	108	103	99	95	88	82
	10-A	105	4125	250	214	200	188	176	167	158	150	143	136	130	125	120	115	107	100
17′-0″	8 R-A	64	1530	90	77	72	68	64	60	57	54	51	49	47	45	43	42	39	36
	10-A	105	3842	226	194	181	170	160	151	143	136	129	123	118	113	109	104	97	90
17′-6″	"	"	3675	210	180	168	158	148	140	133	126	120	115	110	105	101	97	90	84
	12-C	123	4830	276	237	221	207	195	184	174	166	158	151	144	138	132	128	118	110
18′-0″	10-A	105	3528	196	168	157	147	138	131	124	118	112	107	102	98	94	90	84	78
	12-C	123	4662	259	222	207	194	183	173	164	155	148	141	135	130	124	120	111	104
18′-6″	"	"	4477	242	207	194	182	171	161	153	145	138	132	126	121	116	112	104	97
19′-0″	"	"	4275	225	193	180	169	159	150	142	135	129	123	117	112	108	104	96	90
	12-B	140	4769	251	215	201	188	177	167	158	150	143	137	131	125	120	116	108	100
19′-6″	"	"	4602	236	202	189	177	167	157	149	142	135	129	123	118	113	109	101	94
20′-0″	"	"	4460	223	191	178	167	157	149	141	134	127	122	116	112	107	103	95	89
20′-6″	"	"	4264	208	178	166	156	147	139	131	125	119	114	109	104	100	96	89	83
	12-A	160	5104	249	214	199	187	176	166	157	149	142	136	130	125	120	115	107	100
21′-0″	"	"	4935	235	202	188	176	166	157	148	141	134	128	123	118	113	108	101	94
21′-6″	"	"	4773	222	190	178	167	157	148	140	133	127	121	116	111	107	103	95	89
	14-C	170	5074	236	202	189	177	167	157	149	142	135	129	123	118	113	109	101	94
22′-0″	12-A	160	4598	209	179	167	157	148	139	132	125	119	114	109	104	100	96	90	84
	14-C	170	4928	224	192	179	168	158	149	141	134	128	122	117	112	108	103	96	90
22′-6″	"	"	4747	211	181	169	158	149	141	133	127	121	115	110	105	101	97	90	84
23′-0″	"	"	4623	201	172	161	151	142	134	127	121	115	110	105	101	97	93	86	80
	14-B	205	5704	248	212	198	186	175	165	157	149	142	135	129	124	119	114	106	99
23′-6″	"	"	5546	236	202	189	177	167	157	149	142	135	129	123	118	113	109	101	94
24′-0″	"	"	5352	223	191	178	167	157	149	141	134	127	122	116	112	107	103	95	89
24′-6″	"	"	5218	213	183	170	160	150	142	135	128	122	116	111	106	102	98	91	85
	14-A	240	6394	261	224	209	196	184	174	165	157	149	142	136	130	125	120	112	104
25′-0″	"	"	6225	249	214	199	187	176	166	157	149	142	136	130	125	120	115	107	100
25′-6″	"	"	6018	236	202	189	177	167	157	149	142	135	129	123	118	113	109	101	94
26′-0″	"	"	5850	225	193	180	169	159	150	142	135	129	123	117	112	108	104	96	90
	16-C	255	6864	264	226	211	198	187	176	167	158	151	144	138	132	127	122	113	106
26′-6″	"	"	6757	255	219	204	191	180	170	161	153	146	139	133	128	123	118	109	102
27′-0″	"	"	6615	245	210	196	184	173	163	155	147	140	134	128	122	118	113	105	98
27′-6″	"	"	6490	236	202	189	177	167	157	149	142	135	129	123	118	113	109	101	94
	16-B	275	6462	235	201	188	176	166	157	148	141	134	128	122	117	113	108	101	94
28′-0″	"	"	6328	226	194	181	170	160	151	143	136	129	123	118	113	108	104	97	90
28′-6″	"	"	6127	215	184	172	161	152	143	136	129	123	117	112	107	103	99	92	86
29′-0″	"	"	5974	206	177	165	155	146	137	130	124	118	112	108	103	99	95	88	82
	16-A	325	7221	249	214	199	187	176	166	157	149	142	136	130	125	120	115	107	100
29′-6″	"	"	7021	238	204	190	178	168	159	150	143	136	130	124	119	114	110	102	95
30′-0″	"	"	6810	227	195	182	170	160	151	143	136	130	124	118	113	109	105	97	91
30′-6″	"	"	6680	219	188	175	164	155	146	139	132	125	120	115	110	105	101	94	88

rusting. The joints are made mechanically with a 30-ton pressure, to force the metal in the wings of the chord members around the knobs on the web with special 32′-0″ lengths. Table 43 gives total safe loads for various combinations. The standard end bearing is 6″ but this may be varied between 5″

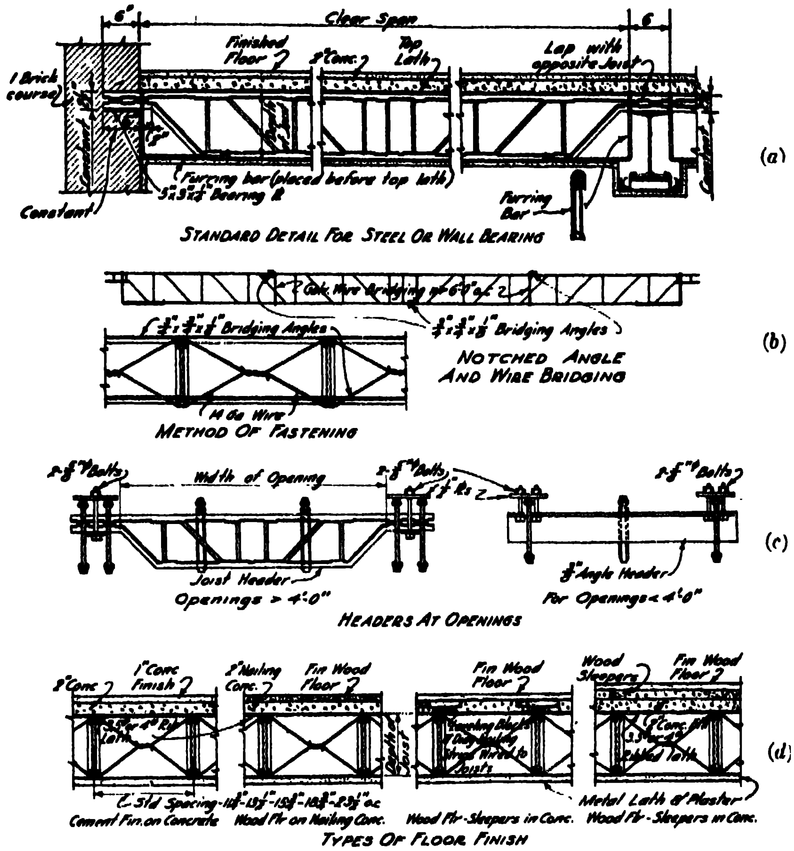


Fig. 183

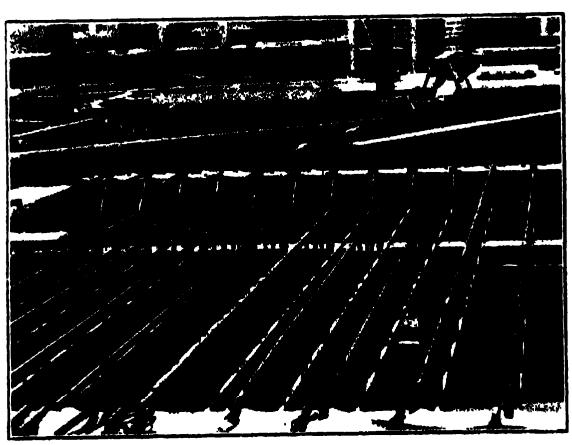


Fig. 184

sections, and to develop the full strength of the component members.

The depths of the joists are from 6″ to 16″ and are made in three types, called " 77," " 66," and " 55." They are available up to and including 30′-0″ spans and 7″, thus allowing a total variation of 4″ in the clear span of a given joist.

The typical floor construction involving the use of these joists is similar to that for other types of this kind. The carrying floor is a 2″ concrete slab,

TABLE 48

Spacing	11¾" C. C.										
Type	55	55	66	66	66	66	77	77	77	77	77
Depth	6"	8"	8"	9"	10"	12"	9"	10"	12"	14"	16"
Joist Wt. Lbs./Ft.	4.1	4.1	5.3	5.3	5.3	5.3	6.7	6.7	6.7	6.7	6.7
Safe Resisting M " #	38600	52700	74200	84300	94500	114700	111500	125300	152800	180400	207900
Clear Span in Feet											
5	510										
6	425										
7	364										
8	318										
9	284	284									
10	259	255									
11	218	232	372								
12	183	212	339	339							
13		196	299	314	314						
14		184	258	293	292	292	365				
15		159	226	256	272	272	337	340			
16		140	198	226	251	255	296	319	319		
17				199	222	240	261	295	300		
18				178	199	227	233	261	283		
19					178	216		235	268		
20					161	195		212	255	255	
21						178			235	242	
22						161			213	232	
23						148			197	222	222
24						136			181	213	212
25										196	204
26										182	196
27										168	189
28										156	181
29											168
30											157

Spacing	13½" C. C.										
Type	55	55	66	66	66	66	77	77	77	77	77
Depth	6"	8"	8"	9"	10"	12"	9"	10"	12"	14"	16"
Joist Wt. Lbs./Ft.	4.1	4.1	5.3	5.3	5.3	5.3	6.7	6.7	6.7	6.7	6.7
Safe Resisting M " #	38600	52700	74200	84300	94500	114700	111500	125300	152800	180400	207900
Clear Span in Feet											
5	444										
6	370										
7	317										
8	278										
9	247	247									
10	225	222									
11	190	202	323								
12	159	185	295	295							
13		171	262	273	273						
14		160	225	256	253	253	317				
15		139	196	223	237	237	294	296			
16		122	172	196	219	222	258	278	278		
17				174	194	209	229	257	262		
18				155	173	198	204	229	247		
19					155	189		206	233		
20					140	170		185	222	222	
21						155			205	211	
22						141			186	202	
23						129			171	193	193
24						118			157	186	185
25										171	178
26										158	171
27										146	165
28										136	157
29											146
30											137

Spacing	15¾" C. C.										
Type	55	55	66	66	66	66	77	77	77	77	77
Depth	6"	8"	8"	9"	10"	12"	9"	10"	12"	14"	16"
Joist Wt. Lbs./Ft.	4.1	4.1	5.3	5.3	5.3	5.3	6.7	6.7	6.7	6.7	6.7
Safe Resisting M " #	38600	52700	74200	84300	94500	114700	111500	125300	152800	180400	207900
Clear Span in Feet											
5	380										
6	317										
7	271										
8	237										
9	212	212									
10	193	190									
11	163	172	278								
12	137	158	253	253							
13		146	223	234	234						
14		137	193	219	217	217	272				
15		119	168	191	203	203	251	253			
16		105	148	168	188	190	221	238	238		
17				149	166	179	195	220	223		
18				133	149	169	174	195	211		
19					133	161		175	200		
20					120	145		159	190	190	
21						132			175	181	
22						120			159	173	
23						110			147	166	166
24						101			135	159	158
25										146	152
26										136	146
27										126	141
28										116	135
29											126
30											117

Spacing	18¾" C. C.										
Type	55	55	66	66	66	66	77	77	77	77	77
Depth	6"	8"	8"	9"	10"	12"	9"	10"	12"	14"	16"
Joist Wt. Lbs./Ft.	4.1	4.1	5.3	5.3	5.3	5.3	6.7	6.7	6.7	6.7	6.7
Safe Resisting M " #	38600	52700	74200	84300	94500	114700	111500	125300	152800	180400	207900
Clear Span in Feet											
5	320										
6	267										
7	228										
8	200										
9	178	178									
10	163	160									
11	137	145	233								
12	115	133	213	213							
13		123	188	197	197						
14		115	162	184	183	183	229				
15		100	142	161	170	170	211	213			
16		88	124	141	157	160	186	200	200		
17				125	140	151	164	185	188		
18				112	125	142	146	164	178		
19					112	136		147	168		
20					102	122		133	160	160	
21						111			147	152	
22						101			134	146	
23						93			124	139	139
24						85			113	134	133
25										123	128
26										114	122
27										106	118
28										98	113
29											106
30											98

Spacing	23½" C. C.										
Type	55	55	66	66	66	66	77	77	77	77	77
Depth	6"	8"	8"	9"	10"	12"	9"	10"	12"	14"	16"
Joist Wt. Lbs./Ft.	4.1	4.1	5.3	5.3	5.3	5.3	6.7	6.7	6.7	6.7	6.7
Safe Resisting M " #	38600	52700	74200	84300	94500	114700	111500	125300	152800	180400	207900
Clear Span in Feet											
5	255										
6	212										
7	182										
8	159										
9	142	142									
10	130	128									
11	109	115	186								
12	92	106	170	170							
13		98	150	157	157						
14		92	129	147	146	146	183				
15		80	113	129	136	136	169	170			
16		70	99	113	126	128	148	160	160		
17				100	111	120	131	148	150		
18				90	100	114	116	131	142		
19					90	108		117	134		
20					81	98		106	128	128	
21						89			117	121	
22						81			107	116	
23						74			99	111	111
24						68			91	107	106
25										98	102
26										91	98
27										84	96
28										78	90
29											84
30											79

supported by ribbed lath with the ribs face down and at right angles to the joists. Temperature rods, $\frac{3}{8}''$ ϕ-24″ o.c. should be placed in the slab longitudinally. Various floor finishes are possible, as shown in Fig. 183 (*d*). When a wood finish flooring is used, 2″ × 2″ screeds are installed and attached by clips. The joists may be spaced $11\frac{3}{4}''$ to $23\frac{1}{2}''$ o.c., as noted in Table 43, thus giving variations to conform with different architectural treatments. Bridging of notched angles and wire, spaced not more than 6′-0″ apart is used to stay the joists laterally, as shown in Fig. 183 (*b*). Figure 184 illustrates a typical installation. This type of construction is adaptable to so-called light loads, and allows easy installation of pipes and conduits between the webs, as do all others of this general type. The use of shelf angle supports on the I beams is eliminated, thereby expediting erection. Erection drawings with identifying shop marks should be furnished for each job.

Section 8c

REINFORCED CONCRETE JOIST CONSTRUCTION IN GENERAL*

120. Characteristics of Its Use.

A form of floor construction which is being used to some extent in connection with steel framed buildings is that which employs a combination of two materials in such a way that a series of concrete joists and intermediate voids are made. Such construction is particularly adaptable to buildings in which the loads are relatively light and well distributed, such as schools, hospitals, apartments and office structures. It has replaced the use of solid slabs to some extent, especially when the spans exceed 12′-0″, as a long span fire-resisting floor without the use of cross beams. A hypothetical illustration of the construction is shown in Fig. 185.

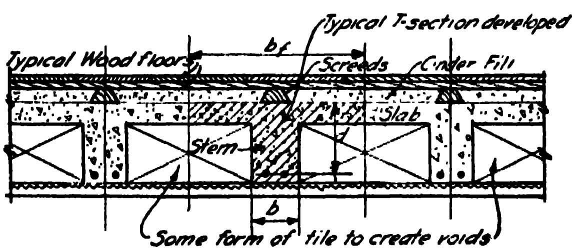


Fig. 185

The voids are made in several ways such as by using (1) metal tiles, either permanent or removable, (2) terra cotta or cinder concrete blocks, (3) gypsum tile blocks, and (4) inverted, removable wood boxes. Figure 186 illustrates some of these methods. The purposes of the tile are:

(1) To create voids in the floor construction, as stated above, and therefore decrease the dead weight and to cut away inactive material. This in turn reduces the sizes of the supporting members.
(2) To provide a form in which the joists and the slab are cast.
(3) To serve as a base and a partial support for the plastered ceiling, which is usually kept flat.

The tile is not counted upon as adding any strength to the floor construction, although the clay tile actually does add somewhat to the ultimate strength of the combination, as considerable arch action is supplied. There is also some natural bond between the two materials, as well as a mechanical bond, due to the projections of the tile scorings into the faces of the joists. The design of such construction is based upon the principles of a T-beam, as developed in the theory of reinforced concrete design. The depth of the joists is often controlled by the shear. A disadvantage sometimes claimed of such construction is that when the ceiling is subjected to a severe fire, the difference in the rates of expansion of the two materials causes the tile to force out of line and consequently distort the floor.

Advantages of this construction are that the strength and stiffness of deep slabs are obtained with a great reduction in the amount of reinforcement and concrete required, the hollow spaces provide good heat and sound insulation, and the structural steel is protected by concrete, — the best material for this purpose. A disadvantage is the fact that the concrete sections are thin and hence are difficult to cast properly, and are not adapted for concentrated loads. These light sections require care in concreting,— in summer to prevent premature drying, and in winter to prevent freezing. The reinforcement in the top slabs is light and is difficult to keep in place during the concreting.

SPECIFICATION CLAUSE†

Combination Floors Concrete floors with permanent blocks or forms of incombustible material with ribs of reinforced concrete between shall conform to the requirements of this act so far as they are applicable, but the blocks or forms shall not be

* Sometimes called "long-span construction" or "ribbed slabs."

† The Building Law of the City of Boston.

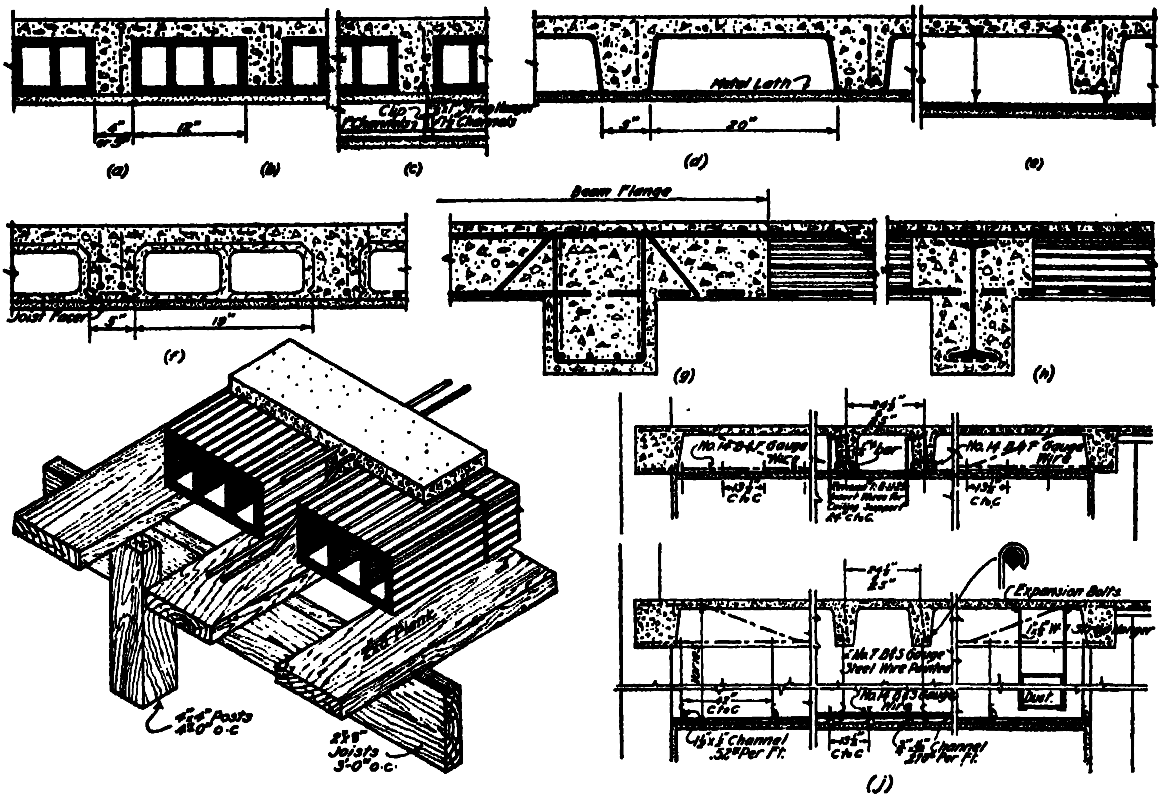


Fig. 186. Types of Ribbed Slabs

(*a*) tile-joist, plastered direct to concrete rib
(*b*) tile-joist with soffit block
(*c*) tile-joist with suspended ceiling
(*d*) stationary metal cores with lathed soffits
(*e*) removable metal cores with suspended ceiling
(*f*) gypsum core
(*g*) concrete girder support for tile-joist slab
(*h*) structural steel girder support for tile-joist slab
(*i*) typical forms for support of ribbed slabs
(*j*) detail for suspended ceiling

assumed as taking stress. If a slab not less than two inches thick above the blocks or forms is cast monolithic with the rib, the rib and slab may be considered as a T-section. If such construction forms a flush ceiling, or if a plastered ceiling on metal lath is suspended below the ribs, the fireproofing for such construction shall be that required for slabs.

The question of what form of ribbed slab construction to use in a particular case depends upon the relative merits of each type, the conditions surrounding the use, and the comparative prices of the materials and labor. As far as dead weight of the floor construction is concerned, the lightest to the heaviest in order are: wood or metal removable forms, permanent metal tile, gypsum blocks, cinder concrete, and terra cotta blocks. The dead weight, while an influencing factor, is not conclusive, and adaptability and economy may control the selection. Quite frequently an estimate will serve as an aid to a decision. The following represents such an estimate made in 1923 in Boston:

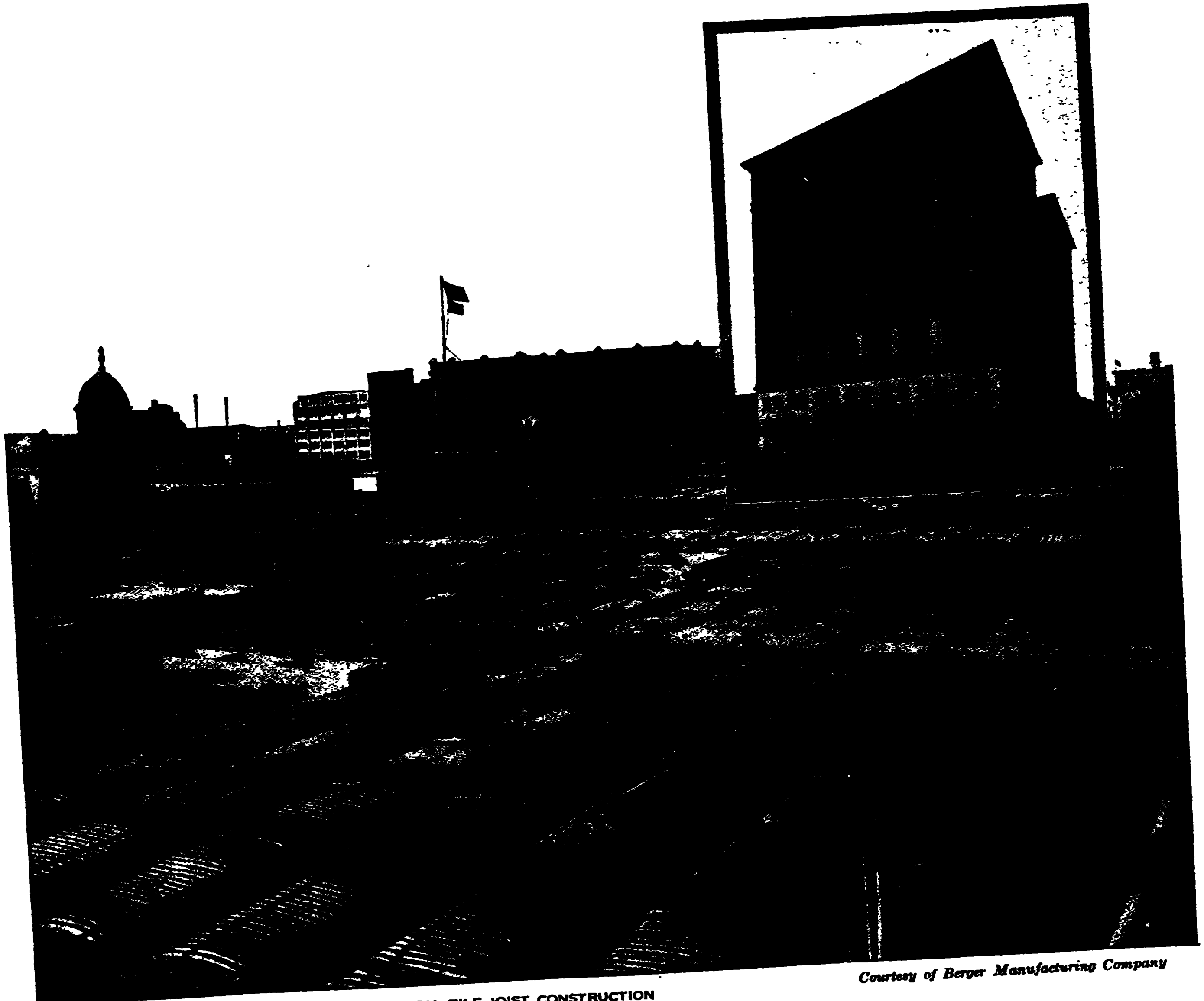
Terra Cotta

	per sq. ft.
6″ T.C.	= $0.200
10% Breakage	= 0.020
Open Forms	= 0.135
Concrete	= 0.165
Soffit Tile	= 0.030
Reinforcement	= 0.090
Trucking	= 0.020
Placing	= 0.030
	$0.690

Metal (Removable)

	(Leased)
6″ Tile	= $0.040
End Caps	= 0.007
Open Forms	= 0.135
Concrete	= 0.165
Reinforcement	= 0.090
Trucking	= 0.015
Placing	= 0.025
Removing	= 0.015
Hangers	= 0.020
Lath	= 0.035
Erecting lath and channels	= 0.115
	$0.662

It should be understood that the above figures may not be the same in all localities, and are not given as a proof of which method is the cheaper, but as an example. They also do not represent the cost of the floor as an entity. The proximity which a building may have to the sources of supply of one kind or another of floor tile may greatly influence the relative costs.



Courtesy of Berger Manufacturing Company

PLATE 13 TYPICAL FRAME SHOWING USE OF METAL TILE JOIST CONSTRUCTION

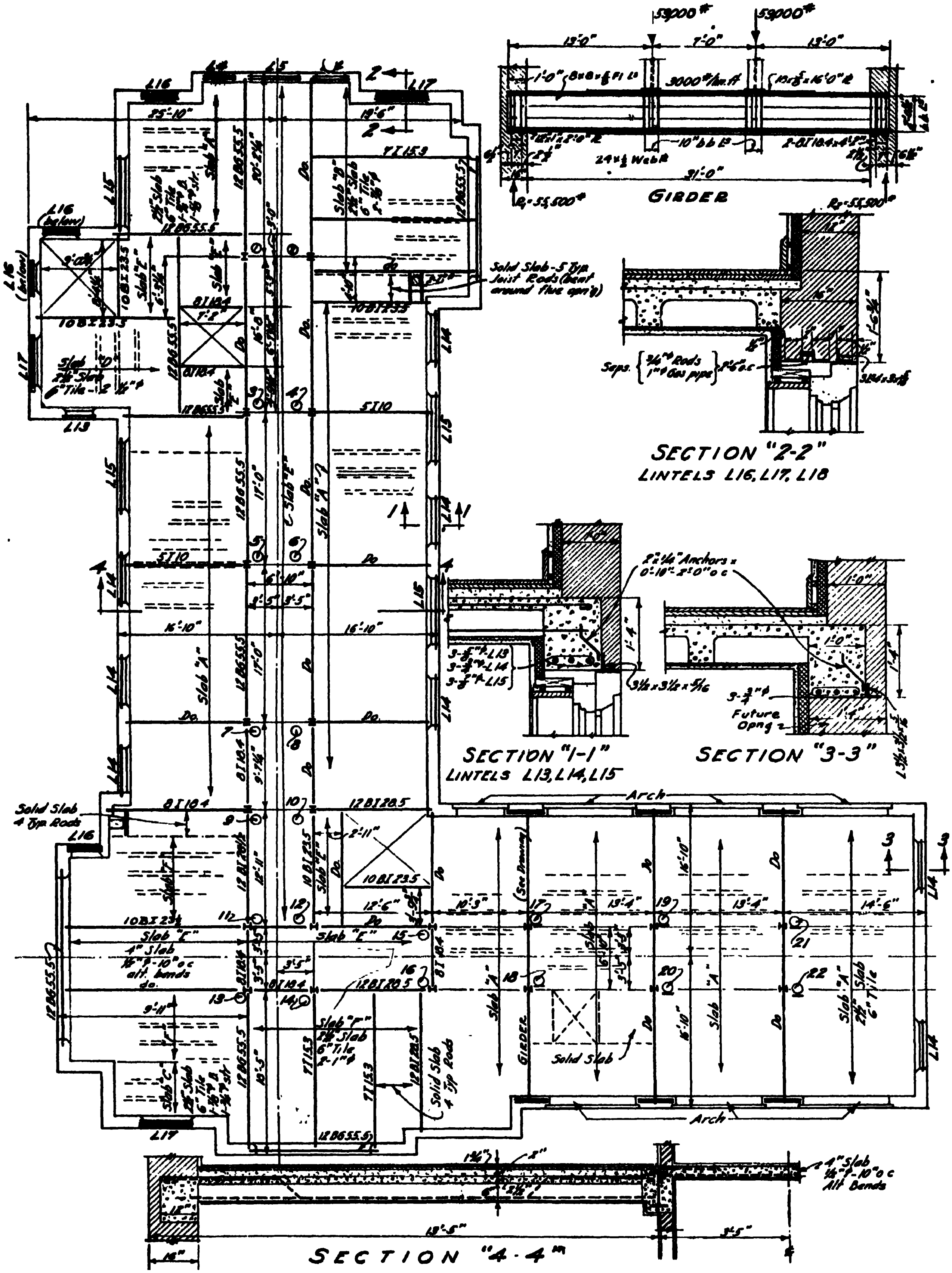


PLATE 14 TYPICAL FRAMING PLAN
METAL TILE-JOIST CONSTRUCTION

121. Variations of the Construction.

Ribbed slab construction may also be used in conjunction with supporting girders of concrete. Such a condition involves no new principles of design if the essentials of the design of the structural floor, as governed by the concrete theory, are understood. Figures 187 (*a*) and (*b*) illustrate methods of support for the joists when terra cotta tile are used. The details when metal tile are employed are similar. The values of the moment coefficients used for the joist design in all cases must be carefully decided, as the construction is not as continuous for steel girders as when concrete girders are used.

Tile-joist construction can be employed with exterior bearing walls of terra cotta, as in Fig. 187 (*c*), or of brick, as in (*d*), or concrete filled terra cotta columns may be used for small buildings, as in (*e*). The details are not as common, however.

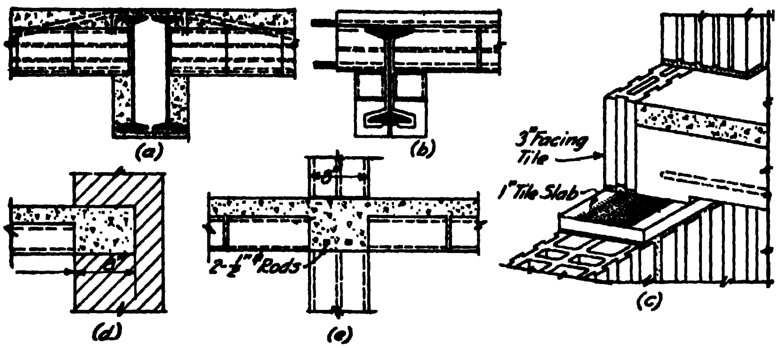


FIG. 187

SECTION 8D

METAL TILE-JOIST CONSTRUCTION

122. Typical Construction.

Figure 188 illustrates a section of the typical floor construction when steel tile are used to form the ribbed slab. Plates 13 and 14 also show some of the characteristics of this construction. In general, the joists are made 5" wide at their soffits, battered to a larger dimension at the base of the slab, and are 20" wide, making the spacing of the joists

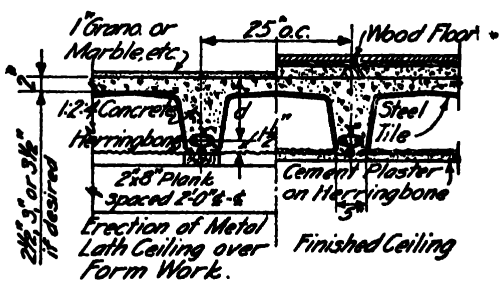


FIG. 188

25" o.c. The slab between the joists is usually made 2" thick, although for heavier loads 2½", 3", and 3½" may be used. The depth of the joists is varied according to the requirements of the design, conforming, however, to the stock depths of the metal tile, which are 4", 6". 8", 10", 12" and 14".* The finish flooring may be varied in the usual way.

Some of the advantages claimed for this construction are:

(1) A **lighter dead load,** the steel tile weighing only about 10% of clay tile, hence
   (*a*) less load on supporting members,
   (*b*) lower freight charges, and
   (*c*) less weight to handle on the job.

(2) A **convenient shape,** — they are slightly tapered, hence
   (*a*) can be nested, shipped, and stored compactly,
   (*b*) can be withdrawn easily,
   (*c*) there is no appreciable leakage of concrete.

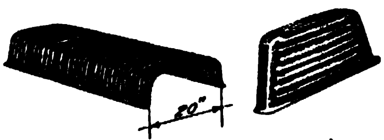
(3) They are not susceptible to **breakage.**

Metal lath must be used to provide a base for the plastered ceiling unless the special types which have a metal plaster base attached to them are employed. This feature should not be lost sight of when comparing the cost of the construction to that of terra cotta blocks. The use of metal tile does not give

* Special depth.

the fire protection to the joists that terra cotta or other blocks do.

The typical tile is shown in Fig. 189 (*a*), which is made of #26 or #14 gauge (depending upon whether the tile is to be permanent or removable) sheet metal, cold pressed into shape. It is usually available in two standard lengths, 30″ and 35″, and it can be made to accommodate any span. Laps of at least one corrugation should be used. The tile



(*a*) (*b*)

Fig. 189

are corrugated or plain, according to the various patented types. For the purpose of preventing the concrete from flowing into the voids, end caps of #28 gauge metal, such as in Fig. 189 (*b*), are used. This feature is a distinct advantage in the saving of concrete, compared with other methods used to stop off the dead space. The end caps are usually left in place in either type of forming.

There are two types of the tile available, as suggested above, namely the permanent and the removable. Figure 190 shows the removable pans. Each offers advantages and the choice must be based upon the particular instance of its use. If the removable type is employed, it can be re-used a number of times (generally every other floor) to form other floors, thereby effecting a saving. Such tile are generally leased to the contractor by the manufacturers, and are for that job only. They are of heavier gauge (usually #14) than the permanent tile in order to have them withstand repeated usage. Being of heavier metal, they insure a more rigid form work, whereas the permanent forms, being of a lighter gauge (usually #26), often sag or become dented by the workmen accidentally walking on them or by the storage of materials on top of them. Since the permanent tile are not salvaged, there is a tendency to make them of lighter metal than they should be. The latter condition adds to the cost of the work, either requiring an attempt to straighten them, which is difficult, or more concrete, as the depressions must be made up by the latter. When permanent tiles are used, the

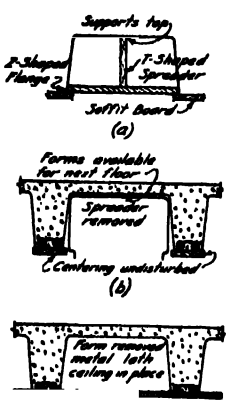


Fig. 190

metal lath for the ceiling is placed on the wood centering before the pans are placed (unless special types are used), so that when the wood supports are removed, the ceiling is ready to be plastered. When removable forms are used, additional labor is required to remove them, and the cost of attaching the metal lath after the wood centering is taken away is a considerable item. However, the sections are nearer to the size designed, and a better inspection of the concrete surfaces is possible. If a suspended ceiling is to be used anyway, there is no waste of a plaster base. There is some variation of opinion as to whether the "joist lines" will show in the finished ceiling or not, and whether any stains will appear later in the ceiling. When clay tile are used as cores, the appearance of the rib lines is quite possible unless soffit blocks are used. When this precaution is taken, the cost increases, due to the increased amounts of materials. The first of these may be caused by the difference in the rates of absorption of the concrete and the plaster, and the second may be produced by the possible rusting of the tile and the mesh, and the infiltration of dust accumulated during the construction. It is the experience of the authors that such blemishes will not appear if careful workmanship, and cement, not lime, plaster are insisted upon, as the bond between cement plaster and concrete is intimate. The removable tile are a real advantage in this respect, however. As a further preventative, the wire suspension clips may be planned so that when the tile is removed and the lath is put into place, it will be 1″ clear of the soffits of the joists. This space incidentally provides a convenient place for conduits and the like. If an entirely flat ceiling is desired (with no beams showing), the removable tile lends itself favorably to the construction, and the typical suspended ceiling may be used. A comparative estimate of the costs for each type will often be an aid in making a decision. The following is a typical example:

Open wood forms	@	.135¢	Same in either case.
Concrete	@	.165¢	
Reinforcing steel	@	.09¢	

	Permanent Forms	Removable Forms
End Caps	$ .007	$ .007
6″ Forms	.08	.04 Lease
Trucking	.01	.015
Placing	.025	.025
Removing	.00	.015
Hangers	.01	.020
Lath	.035	.035
Placing Lath	.020	.00
Erecting Lath and Channels	.00	.115
Total Cost	$0.187	$0.272

Difference .... 8½¢ per sq. ft.

It should be remembered that the above figures will vary in different localities. The difference in the cost of one type from that of the other must be weighed against the relative merits of each method, as discussed above. However, the permanent forms are probably the more used.

123. Typical Design.

The first step in the design of a typical panel is to make a careful estimate of the load to be carried by the floor construction. The proper live load, and the weights of the finish flooring, fill, and ceiling construction are discussed in Chap. 6. It is necessary to anticipate the weight of the slab and the joists in advance of their design (Table 44). A feature of tile-joist construction is that it is described by the depth of the tile and the thickness of the slab, such as 8 + 2, 12 + 2½, etc.

TABLE 44

PROPERTIES OF STEEL TILE-JOIST FLOORS

2" of Concrete over Steel-Tile

Width of Joists in In.	C. to C. of Joists in In.	Size Steel-Tile...	4"	6"	8"	10"	12"	14"
4"	24"	Average weight per square foot..	34.7	40.1	46.0	53.5	61.0	72.6
		Cu. ft. of concrete per sq. ft. of floor	.241	.278	.319	.371	.423	.505
		Core area % of Section....	51.8	58.3	61.7	63.0	63.8	62.2
5"	25"	Average weight per square foot..	36.1	42.3	49.4	57.1	65.3	78.4
		Cu. ft. of concrete per sq. ft. of floor	.251	.293	.342	.396	.452	.537
		Core area % of section	49.8	55.9	59.2	60.3	61.2	59.7

2½" of Concrete over Steel-Tile

Width of Joists in In.	C. to C. of Joists in In.	Size Steel-Tile...	4"	6"	8"	10"	12"	14"
4"	24"	Average weight per square foot..	40.7	46.1	52.0	59.5	67.0	78.6
		Cu. ft. of concrete per sq. ft. of floor	.283	.32	.361	.413	.465	.546
		Core area % of section..........	47.9	54.9	58.6	60.4	61.5	60.3
5"	25"	Average weight per square foot..	42.2	48.3	55.1	63.2	71.4	83.3
		Cu. ft. of concrete per sq. ft. of floor	.293	.335	.382	.438	.495	.579
		Core area % of section.........	46.0	52.6	56.3	58.0	59.2	57.9

The object of the design is to obtain as thin a floor construction as feasible and to obtain the maximum economy. The first investigation should be based upon the requirements for the shear, as it often controls in T-beam design. The distance between the center lines of the girders is established by the column center distances. Usually, the maximum span in the typical floor is selected as a basis for the determination of the joist size, as a common ceiling height is generally desired. The reinforcement in the joists can be varied with the moments in the other spans. In this way the number of sizes of tile is minimized and the form work in general is simplified. The width of the typical girder, or the clear distance between the faces of the fireproofing of the girders, must be assumed. This may be temporarily established by a preliminary design allowing 2" of protection on each face. The projection of the girder is usually made constant wherever possible in order to obtain a uniform length of pans in the majority of cases. The available shear area of a joist is based upon its effective depth and its average width (as the tiles are usually tapered). Table 45 gives such data. The allowable shear is often specified as 60#/□" as a maximum.*

TABLE 45

AVAILABLE SHEAR AREAS OF JOISTS WITH METAL TILE

4 + 2½	31.6□"	10 + 2	75.6□"
4 + 2	28.2	12 + 2½	98.6
6 + 2½	45.5	12 + 2	93.8
6 + 2	41.8	14 + 2½	128.5
8 + 2½	60.8	14 + 2	122.5
8 + 2	56.9	14 + 3	132.7
10 + 2½	80.0		

Illustrative Prob. 114a. Determine the size of joist as governed by shear for a span of 13'-7". L.L. 75#/□'. Typical double wood floor construction.

Assume 6" tile

L.L.	=	75#/□'
Fin. Flr.	=	3
Sub Flr.	=	3
2" Cinder Fill	=	16
6 + 2 Constr.	=	42 (Table 45)
Pl. Ceiling	=	10
T.L.	=	149#/□' say 150

Assume 12" width of F.P. of girder and 12" projection of flange.

Shear span = 13'-7" − (6 + 2 × 1'-0") = 11'-1"

$$\text{Load per ft. on joist} = 150 \times \frac{25}{12} = 312\#$$

$$V = \frac{w \cdot L}{2} = \frac{312 \times 11.08}{2} = 1728\#$$

* The governing code must be consulted in this respect.

$$v = \frac{V}{b \cdot j \cdot d} = \frac{V}{j \cdot A_c} \quad A_c = 41.8\square'' \text{ (Table 45)}$$

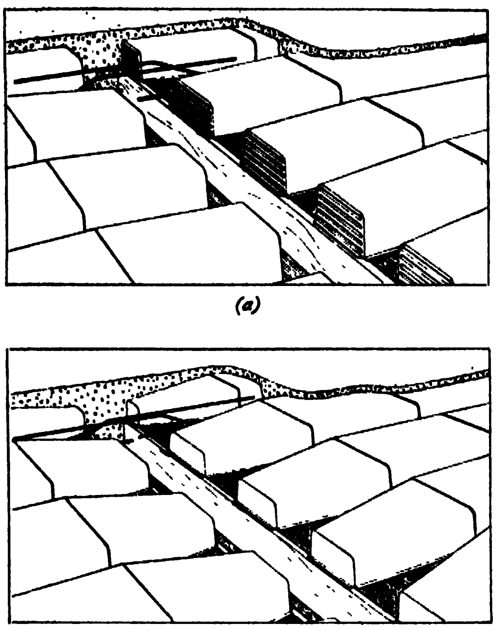
$$v = \frac{1728}{\frac{7}{8} \times 41.8} = 47.3\#/\square'' \quad \text{Shear O.K.}$$

Use 6 + 2 Construction

If the shear exceeds the allowable, the design may be revised by

(1) using a deeper joist,
(2) employing stirrups, or
(3) using special end tile.

The first method is usually uneconomical if there is sufficient strength for the bending moment. The use of stirrups, as suggested in (2), is uncommon and undesirable, because such web reinforcement would necessarily be small and awkward to place and hold in position, although #8 gauge wire stirrups have been used in special instances. For heavy



(a)

(b)

FIG. 191

floors, special tapered tile may be used. These are available for all depths except the 4". They are tapered so that the width of the joist at its ends is 9", thereby doubling the shear-resisting area, approximately. Special end caps are provided for the sizes mentioned. Some companies manufacture a double, tapered tile which slopes down 2" from the standard height and provides an additional thickness of concrete of this amount at the top of the slab as well as at the sides of the joist where it enters the concrete girder. Figure 191 shows an application of such tile.

The calculations for the size of the joist required for the bending moment are made in the usual way. The compression at the supports of the joists will usually be satisfactory, and a slight overstress may even be allowed, as the strengthening action of the flange of the girder is considerable.

Illustrative Prob. 123b. Calculate the area of the reinforcement required and check the size of the joist for moment, for the data of Illustrative Prob. 123*a*. Use Boston Law.

Clear span for moment = 13'-7" − (6") = 13'-1"

Span partially continuous, use $M = \dfrac{w \cdot L^2}{10}$.

$M = 1.2\,w \cdot L^2 = 1.2 \times 312 \times (13.08)^2 = 64{,}100''\#$.

$f_c = 715\#/\square''$, $f_s = 18{,}000\#/\square''$, $K = 125$.

Breadth of flange = c.c. of joists = 25" = b_f.

$$d = \sqrt{\frac{M}{K \cdot b_f}} = \sqrt{\frac{64{,}001}{125 \times 25}} = 4.54''$$

6 + 2 Construction O.K.

($d = 6 + 2 - 1\frac{1}{2} = 6.5''$)

$$A_s = \frac{M}{f_s \cdot j \cdot d} = \frac{64{,}100}{18{,}000 \times \frac{7}{8} \times 6.5} = 0.63\square''$$

Use 1-$\frac{3}{4}''\phi$ *Bent*
1-$\frac{5}{8}''\phi$

The negative moment should be provided for in the customary way.

$-M = 1.2\,w \cdot L^2 = 64{,}100''\#$ (as above)

$-A_s = 0.63\square''$ *Use* 1-$\frac{3}{4}''\phi$ *from bottom of joist, and* 1-$\frac{1}{2}''\phi$ *from joist opposite*

The bend points are usually made at the quarter-points of the respective spans, and the rods are extended to the quarter-points of the adjoining spans.

$$u = \frac{V}{j \cdot d \cdot \Sigma_0} = \frac{1728}{\frac{7}{8} \times 6.5 \times 4.93} = 62\#/\square'' \qquad \textit{Bond O.K.}$$

Temperature rods are used in the slabs, commonly $\frac{3}{8}''\phi$, 18" o.c., in the usual manner.

The design of such construction may be facilitated by the use of tables if the conditions surrounding the design correspond with the table.

When the partitions occur in a direction parallel to the joists, the design must be carefully investigated. If the partition is light, the joist may be strong enough to carry it, as the live load can be omitted on the portion of the floor the partition occupies. Another method is to use a wider joist, but this would usually break up the continuity of the joists with the adjoining spans, which is undesirable. A better method when the partition load is heavy is to leave out a pan at such a point and to use a flat beam of the same depth as the joists. If such a beam is not sufficient, a regular intermediate beam will have to be used.

If the joists do not space out evenly across a panel, a space must be left adjacent to the beam on each side of the panel. This portion of concrete is made the same as that of the joists and should be

TABLE 46

SAFE LOADS UNIFORMLY DISTRIBUTED IN LBS. PER SQ FT.* FOR STEEL TILE-JOISTS

$$M = \frac{w \cdot L^2}{10}$$

5" Joists 25" o.c.

4" Steel Tile + 2" Concrete

Stresses	f_c = 650 lb/□": f_s = 16,000 lb/□"					f_c = 700 lb/□": f_s = 18,000 lb/□"					f_c = 750 lb/□": f_s = 20,000 lb/□"				
RMs	1664	2260	2853	3570	4301	1872	2542	3210	4016	4830	2080	2825	3567	4462	5377
Steel Area Sq. In.	2812	3906	.500	.6406	.7812	2812	.3906	500	6406	7812	.2812	3906	500	6406	7812
Sq. Bar Sizes Straight	1-3/8	1-1/2	1-1/2	1-5/8	1-5/8	1-3/8	1-1/2	1-1/2	1-5/8	1-5/8	1-3/8	1-1/2	1-1/2	1-5/8	1-5/8
Sq. Bar Sizes Bent	1-3/8	1-3/8	1-1/2	1-1/2	1-5/8	1-3/8	1-3/8	1-1/2	1-1/2	1-5/8	1-3/8	1-3/8	1-1/2	1-1/2	1-5/8
Span Length in Feet 5	284	396	514			321	454	583			364	507			
6	186	264	381	440		213	304	394	501		241	341	440		
7	126	185	233	314	386	146	214	279	358	436	167	240	314	475	
8	89	133	179	231	286	104	155	205	265	326	120	176	231	335	366
9	62	94	133	211	219	74	115	195	293	250	88	132	176	265	281
10	44	82	100	135	170	53	87	118	156	195	64	100	135	178	221
11	30	54	76	105	135	38	65	92	123	155	46	75	106	141	176
12		39	59	83	107		49	71	97	125	34	58	82	113	143
13			45	65	86		37	55	78	102		44	65	90	116
14			34	52	69			43	61	83		34	51	73	96
15				40	55			33	49	68			40	59	79
16				31	44				39	54			31	48	64
17					35					44				38	53
18										36					44
19															36

6" Steel Tile + 2" Concrete

Stresses	f_c = 650 lb/□": f_s = 16,000 lb/□"					f_c = 700 lb/□" f_s = 18,000 lb/□"					f_c = 750 lb/□": f_s = 20,000 lb/□"				
RMs	4120	5167	6250	7484	8789	4635	5813	7030	8420	9888		5150	6459	7812	9356
Steel Area Sq. In.	500	6406	7812	9531	1 125	500	6406	7812	9531	1 125		500	6406	7812	9530
Sq. Bar Sizes Straight	1-1/2	1-5/8	1-5/8	1-3/4	1-3/4	1-1/2	1-5/8	1-5/8	1-3/4	1-3/4		1-1/2	1-5/8	1-5/8	1-3/4
Sq. Bar Sizes Bent	1-1/2	1-1/2	1-5/8	1-5/8	1-3/4	1-1/2	1-1/2	1-5/8	1-5/8	1-3/4		1-1/2	1-1/2	1-5/8	1-5/8
Span Length in Feet 10	156	206	258	318	381	180	237	296	363	433		206	268	333	408
11	122	163	206	256	307	142	189	237	292	350		162	215	268	330
12	95	130	167	208	251	113	152	192	239	288		130	174	219	270
13	75	105	136	171	208	90	123	158	197	239		104	142	180	224
14	59	85	111	142	173	72	100	130	165	200		84	116	150	188
15	46	68	91	118	146	57	82	108	138	169		68	96	125	158
16	35	55	75	98	123	45	67	90	116	143		55	79	104	133
17		44	62	82	104	35	55	75	98	122		44	65	88	114
18		35	51	69	88		44	62	83	104		34	54	74	97
19			41	58	75		36	52	70	89			44	62	82
20			33	48	63			42	59	76			36	52	70
21				39	54			35	50	66				43	60
22					45				41	56				35	51
23					37				34	48					43
24										40					36
25										34					

* Weights of floor slab and tile already deducted. To find safe load per sq. ft., deduct finish flooring, fill, ceiling, etc.
In the above and the following tables, heavy lines are drawn for a vertical shearing force producing an average shearing stress of 60 lb. per sq. in. on the concrete. Tapered tile should be used for all loads above and to the right of these lines.

TABLE 46 — *Continued*

8″ Steel Tile + 2″ Concrete

Stresses	f_c = 650 lb/□″: f_s = 16,000 lb/□″					f_c = 700 lb/□″: f_s = 18,000 lb/□″					f_c = 750 lb/□″: f_s = 20,000 lb/□″				
RMs	6799	8250	9038	11,700	13,653	7649	9281	11,180	13,162	15,359	6734	8500	10,312	12,422	14,628
Steel Area Sq. In.	.6406	.7812	.9531	1.125	1.328	.6406	.7812	.9531	1.125	1.328	.500	.6406	.7812	.9531	1.125
Sq. Bar Sizes: Straight	1-⅝	1-⅝	1-¾	1-¾	1-⅞	1-⅝	1-⅝	1-¾	1-¾	1-⅞	1-½	1-⅝	1-⅝	1-¾	1-¾
Sq. Bar Sizes: Bent	1-½	1-⅝	1-⅝	1-¾	1-¾	1-½	1-⅝	1-⅝	1-¾	1-¾	1-½	1-½	1-⅝	1-⅝	1-¾
Span Length in Feet 10	278	348	428	514	604	318	397	488	584		274	359	447	548	
11	221	279	345	416	494	255	319	395	475	563	219	289	361	444	532
12	178	226	282	342	407	206	261	325	390	464	176	235	296	366	439
13	144	186	234	284	340	169	215	270	326	388	143	193	245	305	368
14	118	154	195	238	286	139	179	215	274	328	116	159	203	256	310
15	97	127	163	201	243	114	149	190	232	279	95	133	171	216	264
16	78	106	137	171	207	95	125	161	198	239	77	111	145	184	226
17	64	88	116	146	178	78	105	137	170	206	63	92	122	158	194
18	51	72	98	124	153	64	88	117	146	179	51	77	104	135	168
19	41	61	83	107	133	53	75	100	126	155	41	64	88	116	146
20	33	50	70	91	115	43	62	85	109	136	32	53	75	100	127
21		41	59	78	99	34	52	73	94	118		44	62	86	110
22		33	50	67	86		43	62	82	104		35	54	74	96
23			41	57	75		35	53	70	91			45	64	84
24			34	49	65			44	61	79			37	55	73
25				41	56			37	52	69				47	64
26				34	48				45	60				39	55
27					41				38	52					47
28					35					45					41
29										39					35

10″ Steel Tile + 2″ Concrete

Stresses	f_c = 650 lb/□″: f_s = 16,000 lb/□″					f_c = 700 lb/□″: f_s = 18,000 lb/□″					f_c = 750 lb/□″: f_s = 20,000 lb/□″				
RMs	8455	10,280	12,428	14,640	17,141	9512	11,565	13,981	16,470	19,284	10,569	12,850	15,531	18,300	21,417
Steel Area Sq. In.	.6406	.7812	.9531	1.125	1.328	.6406	.7812	.9531	1.125	1.328	.6406	.7812	.9531	1.125	1.328
Sq. Bar Sizes: Straight	1-⅝	1-⅝	1-¾	1-¾	1-⅞	1-⅝	1-⅝	1-¾	1-¾	1-⅞	1-⅝	1-⅝	1-¾	1-¾	1-⅞
Sq. Bar Sizes: Bent	1-½	1-⅝	1-⅝	1-¾	1-¾	1-½	1-⅝	1-⅝	1-¾	1-¾	1-½	1-⅝	1-⅝	1-¾	1-¾
Span Length in Feet 10	349	437	541	647	767	400	499	615	735		451	561	690	822	
11	279	353	437	525	625	321	402	499	598	713	363	454	560	670	
12	225	286	358	432	516	261	329	410	493	587	298	372	462	554	659
13	184	235	296	360	431	213	272	341	412	492	244	309	386	464	553
14	151	195	248	302	364	176	227	286	347	416	203	259	324	392	469
15	124	163	209	256	309	146	190	242	295	355	169	218	275	334	401
16	102	136	176	218	265	121	160	205	253	305	142	184	235	287	346
17	84	114	150	187	228	101	135	178	217	264	119	157	202	248	300
18	68	96	128	160	197	84	114	151	188	229	100	134	174	215	261
19	56	80	108	138	171	69	97	129	162	200	84	114	150	187	228
20	44	66	92	119	149	57	82	111	141	175	70	97	130	163	200
21	35	55	78	103	130	46	69	95	122	153	58	83	112	143	177
22		45	67	88	113	37	58	82	107	135	48	71	98	125	157
23		36	56	76	98		48	70	93	118	39	60	84	109	138
24			46	65	86		39	59	80	104		50	72	96	122
25			39	55	75			51	69	91		42	62	84	108
26				47	65			42	60	80		34	53	73	95
27				40	56			35	51	70			45	64	84
28					48				44	61			38	55	74
29					41				37	53				47	65
30										46				41	56
31										39				34	50
32															43
33															38

TABLE 46 — *Continued*

12″ Steel Tile + 2″ Concrete

Stresses	f_c = 650 lb/□″: f_s = 16,000 lb/□″					f_c = 700 lb/□″: f_s = 18,000 lb/□″					f_c = 750 lb/□″: f_s = 20,000 lb/□″				
RMs	12,690	14,930	17,620	20,650	23,820	11,400	13,880	16,800	19,840	23,230	12,650	15,410	18,680	22,025	25,810
Steel Area Sq. In.	.7812	.9531	1.125	1.328	1.531	.6406	.7812	.9531	1.125	1.328	.6406	.7812	.9531	1.125	1.328
Sq. Bar Sizes, Straight	1-5/8	1-3/4	1-3/4	1-7/8	1-7/8	1-5/8	1-5/8	1-3/4	1-3/4	1-7/8	1-5/8	1-5/8	1-3/4	1-3/4	1-7/8
Sq. Bar Sizes, Bent	1-5/8	1-5/8	1-3/4	1-3/4	1-7/8	1-1/2	1-5/8	1-5/8	1-3/4	1-3/4	1-1/2	1-5/8	1-5/8	1-3/4	1-3/4
Span Length in Feet															
10	547	653	783	929		483	603	743	890		544	677	834	993	
11	439	529	635	757	883	389	487	603	723	858	439	548	677	809	965
12	359	433	523	624	730	316	399	496	597	711	358	450	558	669	797
13	296	360	437	523	613	259	330	413	499	595	294	373	467	561	669
14	246	296	367	441	519	214	275	347	420	504	245	313	393	474	567
15	206	254	312	377	444	178	232	293	359	431	205	264	334	405	487
16	173	216	266	323	382	149	195	250	307	371	173	224	286	348	420
17	146	184	228	279	328	124	166	215	265	321	146	191	246	301	366
18	123	156	196	241	288	104	141	184	229	279	123	164	212	262	318
19	104	134	170	210	252	88	120	159	190	245	103	140	184	228	279
20	87	112	147	183	221	72	102	136	173	214	87	120	159	199	245
21	73	98	127	160	194	63	86	118	151	188	73	103	138	174	216
22	61	83	110	140	172	48	73	101	132	166	61	88	120	154	192
23	50	70	95	122	151	38	61	87	115	150	50	75	104	135	170
24	41	60	82	107	134		51	75	100	129	40	63	90	118	150
25		50	70	94	118		41	64	87	113		53	78	104	134
26		41	60	82	104			54	76	100		44	67	91	118
27			51	71	92			46	65	88			58	80	105
28			43	62	81				56	77			49	70	93
29				53	71				48	68			41	60	82
30				45	62				41	59				52	73
31					54					50				45	64
32					47					44					56
33															48

14″ Steel Tile + 2″ Concrete

Stresses	f_c = 650 lb/□″: f_s = 16,000 lb/□″					f_c = 700 lb/□″: f_s = 18,000 lb/□″					f_c = 750 lb/□″: f_s = 20,000 lb/□″				
RMs	20,300	23,725	27,250	31,050	35,100	19,430	22,840	26,691	30,655	34,935	21,590	25,380	29,657	34,060	38,810
Steel Area Sq. In.	1.125	1.328	1.531	1.766	2.000	.9531	1.125	1.328	1.531	1.766	.9531	1.125	1.328	1.531	1.766
Sq. Bar Sizes, Straight	1-3/4	1-7/8	1-7/8	1-1	1-1	1-3/4	1-3/4	1-7/8	1-7/8	1-1	1-3/4	1-3/4	1-7/8	1-7/8	1-1
Sq. Bar Sizes, Bent	1-3/4	1-3/4	1-7/8	1-7/8	1-1	1-5/8	1-3/4	1-3/4	1-7/8	1-7/8	1-5/8	1-3/4	1-3/4	1-7/8	1-7/8
Span Length in Feet															
14	420	504	590	684	785	399	484	576	674	778	451	545	649	756	874
15	356	429	505	586	672	337	411	493	577	668	384	464	556	650	751
16	303	367	434	505	581	285	351	424	497	578	327	399	478	560	651
17	260	317	376	439	506	245	297	366	434	504	281	344	416	495	568
18	223	274	326	384	442	211	261	318	378	442	242	299	363	429	499
19	194	238	285	335	390	181	225	278	330	388	210	260	317	384	439
20	166	207	250	295	345	155	195	244	290	341	181	227	278	332	389
21	143	181	219	261	305	134	171	213	256	303	157	199	245	293	338
22	123	158	197	230	271	115	149	187	227	269	137	174	217	260	308
23	106	137	169	204	241	98	129	164	199	239	117	152	191	232	274
24	92	120	149	181	215	84	112	145	177	214	102	134	170	206	246
25	78	104	132	161	193	71	97	127	158	190	88	117	150	184	220
26	66	90	116	143	171	60	84	111	140	171	75	102	133	164	193
27	56	78	102	127	153	50	72	98	124	152	64	89	118	150	178
28	47	68	89	112	137	41	62	85	111	138	54	77	106	131	160
29	38	57	78	99	123	33	52	74	97	121	45	67	91	116	140
30	30	48	67	87	109		43	64	85	108	37	57	80	104	129
31		40	58	77	97		36	55	75	97		49	70	92	116
32		33	50	68	91			47	66	86		41	61	82	104
33			42	59	77			39	57	76		33	52	72	93
34			35	51	67				49	67			45	64	83
35				43	59				42	59			38	55	74
36				37	52				39	51				48	66
37					45					45				41	58
38					39					38				35	51
39															44
40															38

reinforced with a proportionate number of typical joist rods (note spaces on Pl. 14).

Prob. 123c. Check the reinforcement required for the joist spans shown on the left portion of Pl. 14. Use data of Illustrative Prob. 123*a*.

Prob. 123d. Calculate the size of joists required for a span of 16′-0″ between beams. L.L. = 60#/□′. Typical construction. Check your results by Table 46.

Prob. 123e. Provide an arrangement when the typical joist in Illustrative Prob. 123*a* has to carry a 3″ T.C. partition plastered both sides and 9′-0″ high. (Omit the live load for a 1′-0″ width.)

124. Design Examples.

In order to provide a review of the principles involved in the design of steel tile-joist construction, the sizes shown in the panel determined by the columns 33, 34, 44 and 46 on Pl. 14 should be checked up.

Prob. 124a. Design a typical interior panel of steel tile joist construction to carry a L.L. of 60#/□′. Panel 18′-0″ square. Use 8″ tie beam between the columns parallel to the joists. Use Joint Committee Rules.

125. Two-Way Construction.

The usual one-way system may be extended to a two-way construction by the use of what are called "floor domes" or "tin pan" construction. These are similar to the common steel tile except that they are square and have four closed sides instead of two. They are made of #16 gauge metal and are either 6″ or 8″ high. Their size is 20″ × 20″ effective so that the joists are 25″ o.c. each way. Such construction is not common for the usual floor, but they have been used in a modified flat slab construction which is sometimes called the Grid System. The reason that the two-way system is not common for ordinary floors is that not enough advantages over the one-way construction are gained, and furthermore a solid wood form is required for its construction support. However, the design may be made in a manner similar to that of two-way solid slabs (Art. 145), embodying the special features of the tile-joist framing, as described for one-way framing.

Section 8E

CLAY TILE-JOIST CONSTRUCTION

126. Typical Construction.

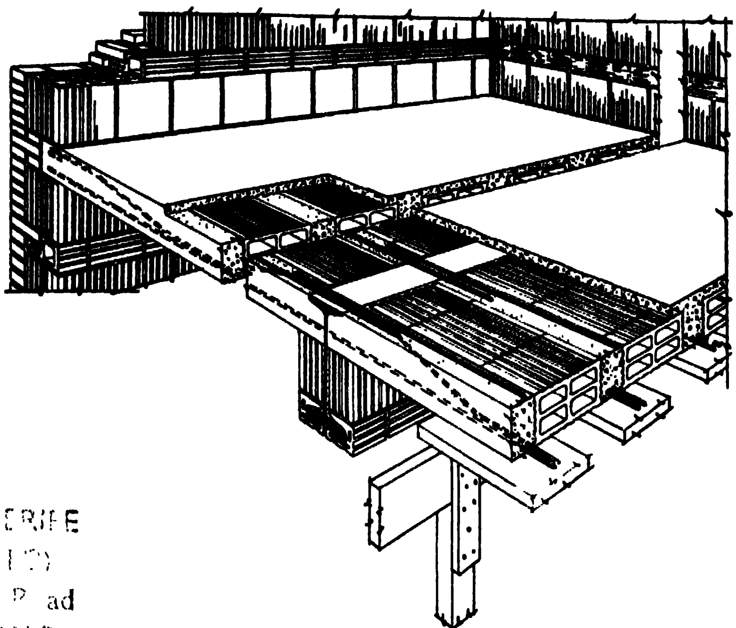
Another form of tile-joist or ribbed slab construction employs terra cotta blocks to produce the voids between the joists and is similar to that discussed in Sect. 8D. Figure 192 * shows a typical view of this construction. Compared with metal tile-joist construction, it offers the advantage of a more fire-resisting structural floor, as the clay tile remain a permanent part of the floor, and add considerable stiffness. Another advantage is that the terra cotta blocks are not as easily deformed by carelessness as steel tile are. Disadvantages sometimes claimed are that the terra cotta blocks get out of line before the concrete is cast, that they are more apt to break, and that the dead weight of the floor is greater than when steel tile are used, thereby increasing the cost of the floor and the supporting frame. It is more difficult to seal the joints and prevent leakage of concrete, and the blocks tend to absorb water from the concrete. If the ceiling of such construction is subjected to a severe fire, the unequal expansion of the concrete and the terra cotta may cause the webs and walls of the blocks to shear and this would allow the bottoms of the blocks to fall out.

Figure 193 illustrates a typical section of the construction of this floor. The joists are usually made 4″ wide, and since the common structural terra cotta blocks are 12″ × 12″ in plan, the joists are 16″ o.c., although 5″ and 6″ joists may be used. The slab is generally made 2″ thick, although 2½″ and 3″ may be used. The depth of the joists is varied according to the loads and the spans, to conform to the standard thicknesses of the blocks which are 3″ to 10″ inclusive by 1″ increments, 12″ and 15″ deep, the 4″, 6″, 8″, 10″ and 12″ blocks being more common. The semi-porous grade of terra cotta is usually employed as it is more fire-resistant than the hard burned grade, although it is not quite as strong. At the faces of the girders, the ends of the tile should be closed off with end tile, cardboard, or plaster of Paris, to prevent the concrete from entering the "dead spaces" or voids. Figure 193 shows two alternate details at the bottoms of the joists, — that in (*a*), a plain concrete soffit, and that in (*b*), the use of soffit tile. The latter is preferable in order to prevent the "joist lines" from showing in the finished ceiling, as discussed in Art. 122. Such blemishes are more liable to occur in this construction than when metal tile is used, as the terra cotta absorbs the moisture from the plaster at a different rate than the concrete. The soffit tile is 1″ thick and is scored on both sides to secure bond between the concrete and the tile and also between the tile and the plaster. An important feature from a

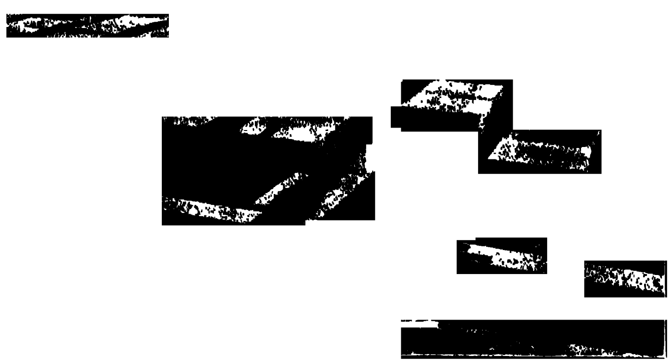
* Courtesy of the National Fireproofing Co.

design standpoint is that the effective depth of the joist is decreased 1″ if soffit tile are used. This is a disadvantage from the standpoint of economy but it is very desirable from a construction point of view.

3″	14#	8″	30#
4	16	9	33
5	20	10	35
6	22	12	40
7	27	15	48



(a)



(b)

Fig. 192*

127. Typical Design.

The design of terra cotta tile-joist construction is quite similar to that discussed when metal tile is used (Art. 123). As in any case, the dead load corresponding to the weight of the floor construction must be determined. The loads imposed by the finish flooring, the fill if used, and the ceiling are calculated in the usual manner. The weights of terra cotta blocks are as follows:

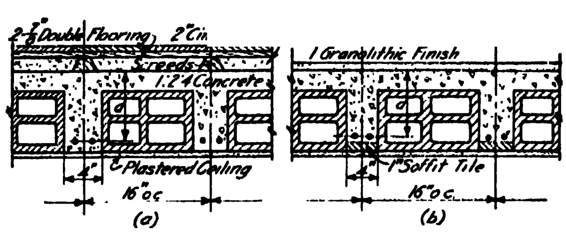


Fig. 193

* Courtesy of National Fireproofing Co.

These values are used in obtaining the weight of the structural floor. However, the weight of the terra cotta blocks, the slab and the joists is more conveniently expressed as a definite load per square foot of floor area. Table 47 gives these values in a usable form.

Stirrups are awkward to place and to hold in position, and only very small sectional areas of stee are required. This method is not advised except in extreme cases. The second method provides additional shear resisting area by using 8″ × 12″ tile, as shown in plan in Fig. 194 (*b*). The number of such tile required at the ends of the joists may be found by determining the distance from the support where the ordinary joist section is sufficient to resist the shear. This method provides a more definite resistance to shear, and also offers additional resistance to compression adjacent to the supports. The third method, that of increasing the depth of the joists, is

TABLE 47

WEIGHT OF T.C. TILE-JOIST CONSTRUCTION

(Lbs. per Sq. Ft.)

Thickness of Tile...	3 in.	4 in.	5 in.	6 in.	7 in.	8 in.	9 in.	10 in.	12 in.	15 in.
2-in. Concrete Top — 4-in. Joists, 16-in. on centers										
Weight per sq. ft. of floor area..	45 lb.	50 lb.	55 lb.	60 lb.	65 lb.	70 lb.	75 lb.	80 lb.	90 lb.	105 lb
Cu. ft. concrete per sq. ft. floor .	0.229	0.250	0.271	0.292	0.313	0.333	0.354	0.375	0.417	0.479
2½-in. Concrete Top — 4-in. Joists, 16-in. on centers										
Weight per sq. ft. of floor area .	51 lb.	56 lb.	61 lb.	66 lb.	71 lb.	76 lb.	81 lb.	86 lb.	96 lb.	111 lb.
Cu. ft. concrete per sq. ft. floor..	0.271	0.292	0.313	0.333	0.354	0.375	0.396	0.417	0.458	0.521

The first step in the design is to determine the depth of the joist required by shear, as T-beams are often controlled in size by this stress. The effective width of the shear resisting area is usually based upon the following:

SPECIFICATION CLAUSE (J. C.)

> The shearing stress in tile-and-concrete-beam construction shall not exceed that in beams or slabs with similar reinforcement. The width of the effective section for shear, as governing diagonal tension, shall be taken as the thickness of the concrete web plus one-half the thickness of the vertical webs of the tile.

This width is indicated in Fig. 194 (*a*). The reason for this assumption is that the terra cotta blocks are scored, so that the concrete contained within the scoring as well as a part of the outside walls of the blocks is effective in resisting shear. These walls are usually ¾″ thick in the common blocks. The allowable shearing stress for such areas is often specified as 60#/□″.*

If the actual shear exceeds the allowable, it may be reduced to within allowable limits by the following methods:

(1) Web reinforcement in the form of small vertical stirrups may be used.
(2) Special narrow tile at the ends of the joists may be employed.
(3) The depth of the joist may be increased.

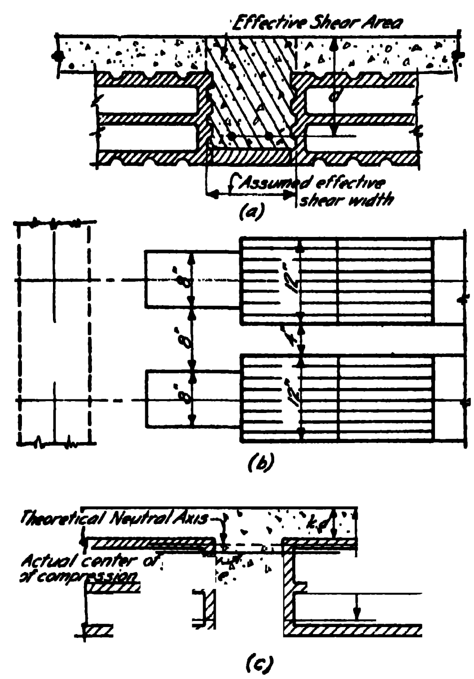


FIG. 194

* Consult the governing code.

the most commonly used in commercial practice, as the special tile required in method (2) are often difficult to obtain, and special formwork is often necessary to support them. The increased depth is also of value in resisting compression, and the required area of the reinforcement will be proportionately less.

When a size of joist has been selected which is satisfactory for the requirements of shear, the depth of the joist should be checked to show sufficient resistance to the bending moment. The tensile reinforcement should then be selected. Since the span in the design of members in monolithic construction is usually taken as the clear distance between the supports, that for the moment calculations in tile-joist construction may be taken as the distance between the faces of the girder encasement. The breadth of the flange of the T-joist section is the spacing of the joists. For deep joists and 2″ slabs, the neutral axis of the section may theoretically be located below the flange, but the theoretically exact formulas are seldom employed in commercial design, and particularly in joist design, where the loads and sections are relatively small. The location of the neutral axis, as far as the actual compression resistance is concerned, compared with the theoretical position, is illustrated in Fig. 194 (*c*). The bend points of the rods and their lengths of embedment to develop bond are determined in the customary manner.

In many cases, the sizes of the joists, together with their reinforcement, may be selected from tables, if the conditions surrounding the design correspond with the table. Tables 48 and 49 are representative of such forms of determining the required sizes.

The design of the supporting beams and girders is similar to that discussed in Art. 123. It is claimed erroneously that no temperature rods are required in the slab when clay tile are used, as the heat is not transmitted so as to accumulate at points, as is the case when metal tile are used. It is wise to provide $\frac{3}{8}''\ \phi$ rods, 24″ o.c. in all cases.

Prob. 127a. Substitute for the sizes of the joists shown on Pl. 14 for the typical interior panel for the shear requirements, using terra cotta tile and 4″ joists.

Prob. 127b. Proportion the joist for the bending moment for the data of Illustrative Prob. 123*a*.

Prob. 127c. What sizes of joists and reinforcement are required to carry a live load of 75#/□′ on an 18′-0″ span? Typical construction with soffit tile. Use J. C. Rules.

Prob. 127d. Check the sizes and reinforcement for the remaining joist spans shown on Pl. 14 for terra cotta tile joist construction. Use data similar to Illustrative Prob. 123*a*. Check by the use of Table 49.

Prob. 127e. Design the typical supporting girder for Prob. 127*c*.

Prob. 127f. Design the girders shown on Pl. 14 for the data of Prob. 127*d*.

128. Two-Way Construction.

The one-way system of terra cotta tile-joist construction may be extended to two-way framing, if desirable. Figure 195 illustrates the typical construction. The joists are usually made 4″ wide with skew and key terra cotta blocks between them, as shown, making the spacing of the joists 28″ o.c. in each direction. The typical 2″ slab, finish flooring, and ceiling construction are used as before. Cross beams should be used on all column center-lines.

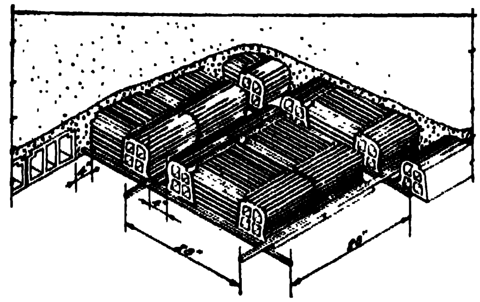


FIG. 195

This construction is not as commonly used, as the advantages gained are not sufficient in comparison with one-way systems, particularly for long spans, as the percentage of " inert " material is less. Some terra cotta companies have discontinued the manufacture of the special blocks required for this work. Since the load is carried in two rectangular directions, the depth of the joists, and consequently the thickness of the floor, is less for a given load and span. The proportions of the load carried in each direction may be established by the principles outlined in Art. 107, and the loads carried by the supporting girders or walls on the four sides of the panels may be determined in the manner discussed in Art. 145. The design of the individual joists in each respective direction is similar to those in one-way construction. Table 50 gives data which will often save time and calculations.

A patented type of floor construction is the Republic Two-Way system,* as illustrated in Fig. 196. In general, it consists of rectangular terra cotta blocks supplemented by channel and soffit blocks in such a way as to produce joists in two directions at right angles to each other, as shown. These blocks automatically space themselves and the channel sections close the ends of the rectangular pieces, thereby preventing the inflow of concrete into the voids. The soffit blocks provide an all tile ceiling base and hence eliminate the tendency of " joist-lines " showing. They also aid in protecting the concrete, and two-coat plaster work may be used. It is claimed that the salvage of the form lumber is high because practically no concrete is allowed to come in contact with it.

The lower reinforcing rods are placed upon the sides of

* Republic Fireproofing Company, Inc., New York City.

TABLE 48

SAFE UNIFORMLY DISTRIBUTED LOADS FOR TERRA COTTA TILE-JOIST CONSTRUCTION

2" Concrete Top

f_c = 650 lb. per sq. in.
f_s = 16,000 lb. per sq. in.

$\frac{Ec}{Es} = \frac{1}{15}$

$\frac{3}{4}$" of concrete below reinforcemen
4" concrete joists 16" on centers

Bending Moment	Total Safe Loads (Dead & Live)											
$\frac{W \cdot L}{12}$	150	165	180	195	210	225	240	260	300	335	375	450
$\frac{W \cdot L}{10}$	125	135	150	160	175	185	200	220	250	280	310	375
$\frac{W \cdot L}{9}$	110	120	135	145	155	170	180	195	225	250	280	335
$\frac{W \cdot L}{8}$	100	110	120	130	140	150	160	175	200	225	250	300
Span 6'-0"						3/.19	3/.20	3/.22	3/.26	3/.29	3/.32	3/.39
" 7'-0"		3/.19	3/.21	3/.23	3/.24	3/.26	3/.28	3/.32	3/.35	3/.38	3/.44	4/.42
" 8'-0"	3/.23	3/.25	3/.27	3/.30	3/.32	3/.34	3/.37	3/.40	3/.46	4/.41	4/.46	4/.55
" 9'-0"	3/.29	3/.32	3/.35	3/.37	3/.39	3/.41	3/.43	4/.40	4/.46	4/.52	4/.58	5/.57
" 10'-0"	3/.36	3/.39	3/.43	3/.46	4/.40	4/.43	4/.46	4/.50	4/.57	5/.53	5/.59	5/.71
" 11'-0"	3/.43	3/.47	4/.42	4/.45	4/.48	4/.52	4/.55	4/.61	5/.57	5/.64	5/.72	6/.73
" 12'-0"	4/.41	4/.45	4/.49	4/.53	4/.58	5/.51	5/.55	5/.60	5/.68	6/.65	6/.72	7/.78
" 13'-0"	4/.48	4/.53	4/.58	5/.52	5/.56	5/.60	5/.64	5/.70	6/.68	6/.77	7/.76	8/.80
" 14'-0"	4/.56	5/.51	5/.56	5/.60	5/.65	5/.69	6/.63	6/.69	6/.79	7/.79	8/.78	9/.85
" 15'-0"	5/.53	5/.58	5/.64	5/.69	6/.63	6/.68	6/.72	6/.79	7/.81	8/.81	8/.89	10/.88
" 16'-0"	5/.60	5/.68	5/.72	6/.67	6/.72	6/.77	7/.74	7/.81	8/.81	9/.84	9/.93	12/.83
" 17'-0"	5/.68	6/.64	6/.70	6/.75	6/.81	7/.78	7/.83	8/.80	9/.84	10/.84	10/.94	12/.93
" 18'-0"	6/.65	6/.72	6/.78	7/.76	7/.82	8/.77	8/.82	8/.90	9/.94	10/.95	12/.87	15/.83
" 19'-0"	6/.73	6/.80	7/.78	7/.84	8/.80	8/.86	9/.84	9/.92	10/.95	12/.87	12/.97	15/.93
" 20'-0"	6/.81	7/.79	8/.76	8/.82	8/.89	9/.87	9/.93	10/.91	12/.86	12/.97	15/.86	15/1.03
" 21'-0"	7/.79	8/.77	8/.85	8/.91	9/.89	10/.86	10/.92	12/.83	12/.95	15/.85	15/.94	
" 22'-0"	8/.77	8/.84	9/.84	9/.91	10/.88	10/.94	12/.83	12/.91	15/.83	15/.93	15/1.04	
" 23'-0"	8/.84	9/.84	9/.91	10/.89	10/.96	12/.85	12/.91	12/.99	15/.90	15/1.02		
" 24'-0"	9/.84	9/.92	10/.90	12/.80	12/.87	12/.93	12/.99	15/.87	15/.99			
" 25'-0"	9/.91	10/.89	12/.81	12/.87	12/.94	12/1.0	15/.86	15/.94	15/1.07			

The upper figures in tables denote the depth of tile; the lower figures indicate the area of reinforcing steel required in each concrete joist. Thickness of floor = depth of tile + 2" of concrete top.

This table and the one on the following page are so arranged that they can be used for floor slabs freely supported at both ends, semi-continuous, or continuous.

For slabs freely supported at both ends (simple span), use load given opposite $\frac{W \cdot L}{8}$.

For slabs freely supported at one end and continuous over other support, use loads given opposite $\frac{W \cdot L}{9}$, or if building code permits, $\frac{W \cdot L}{10}$.

For slabs continuous over both supports, use loads given opposite $\frac{W \cdot L}{10}$, or if building code permits, $\frac{W \cdot L}{12}$.

For semi-continuous and continuous spans proper reinforcement must be provided in top of slab over support to take care of negative bending moment. Where heavy loads and short spans are encountered, the vertical and longitudinal shear must be investigated.

TABLE 49

SAFE UNIFORMLY DISTRIBUTED LOADS FOR TERRA COTTA TILE-JOIST CONSTRUCTION

2" Concrete Top

f_c = 700 lb. per sq. in.
f_s = 18,000 lb. per sq. in.
Shear 60 lb. per sq. in.

$\frac{Ec}{Es} = \frac{1}{15}$

$\frac{3}{4}$ in. of concrete below reinforcement
4 in. concrete joists 16 in. on centers

Bending Moment	Total Safe Loads (Dead & Live)											
$\frac{W \cdot L}{12}$	150	165	180	195	210	225	240	260	300	335	375	450
$\frac{W \cdot L}{10}$	125	135	150	160	175	185	200	220	250	280	310	375
$\frac{W \cdot L}{8}$	100	110	120	130	140	150	160	175	200	225	250	300
Span 6'-0"						3/ /.18	3/ /.20	3/ /.21	3/ /.25	3/ /.28	4/ /.24	4/ /.29
" 7'-0"	3/ /.17	3/ /.18	3/ /.20	3/ /.22	3/ /.23	3/ /.25	3/ /.27	3/ /.29	3/ /.33	4/ /.30	4/ /.33	5/ /.33
" 8'-0"	3/ /.22	3/ /.24	3/ /.26	3/ /.28	3/ /.30	3/ /.33	3/ /.35	3/ /.38	4/ /.34	4/ /.39	5/ /.35	6/ /.36
" 9'-0"	3/ /.28	3/ /.30	3/ /.33	3/ /.36	3/ /.39	3/ /.41	3/ /.44	4/ /.38	4/ /.43	5/ /.40	5/ /.45	6/ /.46
" 10'-0"	3/ /.34	3/ /.37	3/ /.41	3/ /.44	4/ /.38	4/ /.40	4/ /.43	4/ /.47	5/ /.44	5/ /.50	6/ /.47	7/ /.49
" 11'-0"	3/ /.41	4/ /.36	4/ /.39	4/ /.42	4/ /.45	4/ /.49	4/ /.52	5/ /.47	5/ /.54	6/ /.51	7/ /.50	8/ /.53
" 12'-0"	4/ /.39	4/ /.43	4/ /.46	4/ /.50	4/ /.54	5/ /.48	5/ /.51	5/ /.56	6/ /.54	7/ /.53	8/ /.52	9/ /.56
" 13'-0"	4/ /.45	4/ /.50	4/ /.54	5/ /.49	5/ /.52	5/ /.56	5/ /.60	5/ /.66	6/ /.64	7/ /.62	8/ /.62	9/ /.66
" 14'-0"	4/ /.53	5/ /.48	5/ /.52	5/ /.56	5/ /.61	5/ /.65	6/ /.59	6/ /.65	7/ /.64	8/ /.64	9/ /.64	10/ /.70
" 15'-0"	5/ /.50	5/ /.55	5/ /.60	5/ /.65	6/ /.60	6/ /.64	6/ /.68	6/ /.74	7/ /.74	8/ /.74	10/ /.67	12/ /.67
" 16'-0"	5/ /.57	5/ /.62	5/ /.68	6/ /.63	6/ /.68	6/ /.72	6/ /.77	7/ /.73	8/ /.74	9/ /.75	10/ /.76	12/ /.77
" 17'-0"	5/ /.64	6/ /.60	6/ /.65	6/ /.71	6/ /.76	7/ /.71	7/ /.76	7/ /.83	8/ /.84	9/ /.85	10/ /.85	12/ /.86
" 18'-0"	6/ /.61	6/ /.67	6/ /.73	6/ /.79	7/ /.74	7/ /.80	7/ /.85	8/ /.82	9/ /.84	10/ /.86	12/ /.81	15/ /.78
" 19'-0"	6/ /.68	6/ /.75	7/ /.71	7/ /.77	7/ /.83	7/ /.89	8/ /.84	8/ /.92	9/ /.94	10/ /.96	12/ /.90	15/ /.88
" 20'-0"	6/ /.75	7/ /.72	7/ /.79	7/ /.85	7/ /.92	8/ /.87	8/ /.93	9/ /.91	10/ /.95	12/ /.90	12/ /1.00	15/ /.97
" 21'-0"	7/ /.72	7/ /.80	7/ /.87	8/ /.83	8/ /.90	8/ /.96	9/ /.92	9/ /1.00	10/ /1.04	12/ /.99	15/ /.89	15/ /1.06
" 22'-0"	7/ /.79	7/ /.87	8/ /.84	8/ /.91	8/ /.98	9/ /.95	10/ /.92	10/ /1.00	12/ /.97	12/ /1.09	15/ /.98	
" 23'-0"	7/ /.87	8/ /.85	8/ /.92	8/ /1.00	9/ /.97	10/ /.94	10/ /1.00	10/ /1.09	12/ /1.05	15/ /.96	15/ /1.06	
" 24'-0"	8/ /.84	8/ /.92	8/ /1.00	9/ /.98	10/ /.95	10/ /1.02	10/ /1.09	12/ /1.00	12/ /1.15	15/ /1.04	15/ /1.16	
" 25'-0"	8/ /.91	8/ /1.00	9/ /.98	10/ /.96	10/ /1.03	10/ /1.11	12/ /1.00	12/ /1.09	15/ /1.01	15/ /1.13		

For notes, see Table 48.

TABLE 50

TWO-WAY COMBINATION LONG SPAN FLOORS

Data for Designing Purposes

The load tables are for general information only, as each particular operation should be designed in accordance with actual conditions.

2″ Concrete Top

f_c = 700 lb. per square inch
f_s = 18,000 lb. per square inch

$$M = \frac{W \cdot L}{12}$$

¾″ of concrete below reinforcement
4″ concrete joists 28″ on centers

W Part of load taken in each direction

		Total Safe Load (Dead & Live)																		
Total Load per Square Foot Rectangular Panel whose Sides have Ratio of	1 : 1.50					104	110	117	124	130	136	143	150	156	162	169	175	182	188	195
	1 : 1.40				103	110	117	123	130	137	144	151	158	165	171	178	185	192	199	205
	1 : 1.35				106	113	120	127	134	141	148	155	162	169	176	183	190	197	204	211
	1 : 1.30			102	109	116	123	131	138	145	152	160	167	174	181	189	196	203	210	218
	1 : 1.25			106	114	121	129	136	144	152	159	167	174	182	190	197	205	212	220	227
	1 : 1.20		103	111	119	127	135	143	151	159	167	175	183	191	198	206	214	222	230	238
	1 : 1.15	104	108	117	125	134	142	150	158	167	175	183	192	200	208	217	225	233	242	250
	1 : 1.10	110	114	123	132	141	149	158	167	176	184	193	202	211	219	228	237	245	255	263
	1 : 1.00	125	130	140	150	160	170	180	190	200	210	220	230	240	250	260	270	280	290	300
Short Span	16′-0″	3	3	4	4	4	4	4	4	4	4	5	5	5	5	5	5	5	5	5
	17′-0″	4	4	4	4	4	4	4	4	5	5	5	5	5	5	5	5	5	6	6
	18′-0″	4	4	4	4	4	4	5	5	5	5	5	5	5	5	6	6	6	6	6
	19′-0″	4	4	4	4	5	5	5	5	5	5	5	6	6	6	6	6	6	6	6
	20′-0″	4	4	5	5	5	5	5	5	5	6	6	6	6	6	6	6	6	7	7
	21′-0″	4	5	5	5	5	5	5	6	6	6	6	6	6	6	6	7	7	7	7
Thickness of Tile	22′-0″	5	5	5	5	5	5	6	6	6	6	6	6	7	7	7	7	7	7	7
	23′-0″	5	5	5	5	6	6	6	6	6	6	7	7	7	7	7	7	7	7	8
	24′-0″	5	5	5	6	6	6	6	6	6	7	7	7	7	7	7	8	8	8	8
	25′-0″	5	5	6	6	6	6	6	7	7	7	7	7	7	8	8	8	8	8	8
	26′-0″	6	6	6	6	6	6	7	7	7	7	7	8	8	8	8	8	8	9	9
	27′-0″	6	6	6	6	7	7	7	7	7	7	8	8	8	8	8	8	9	9	9
	28′-0″	6	6	6	7	7	7	7	7	8	8	8	8	8	8	9	9	9	9	9
	29′-0″	6	6	7	7	7	7	7	8	8	8	8	8	9	9	9	9	9	10	10
	30′-0″	6	7	7	7	7	7	8	8	8	8	9	9	9	9	9	9	10	10	10

the soffit blocks, hence fixing their position in the short direction. The upper bars are placed so that they are supported by the lower ones, running in the opposite direction. Distinct advantages are gained in the small depth of floor required, as it is stiffer than a one-way system because the deflection in one direction is restrained by the resistance in the other direction. A load produces reactions at every

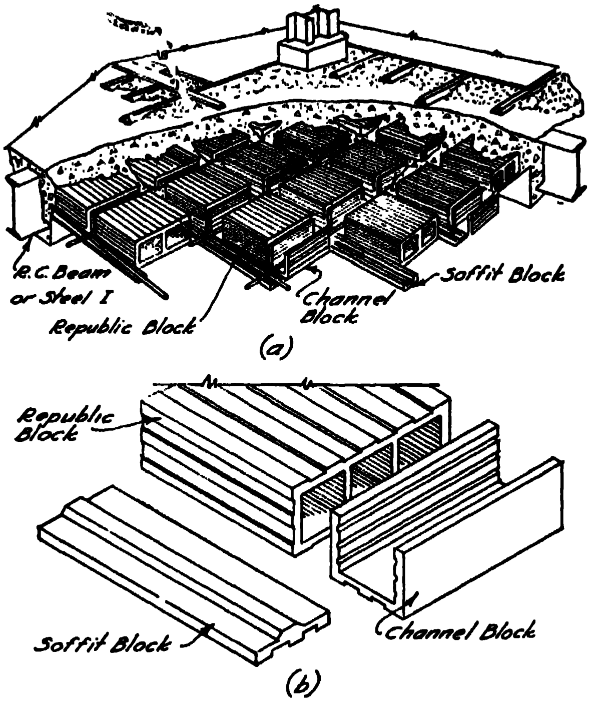


FIG. 196

point of support in a two-way system and hence moving loads are better distributed. In this way the loads on walls and footings are respectively lighter and some economy can be effected. **The terra cotta blocks are counted upon for no strength,** which is additional protection. The concrete is stressed in two directions. Due to the restraint, a higher safe stress might be possible, but since this is practically indeterminate, **the usual allowable compressive stress in the concrete should be used.** However, the live load per square foot may be reduced (Art. 93).

The rectangular blocks are 12″ × 12″ in plan and come in 4″, 5″, 6″, 8″, 10″ and 12″ depths. The soffit blocks and channels are about 5″ wide and the distances center to center of joists are then about 17″. Joists are doubled under partitions. The system may be used with either a structural steel or reinforced concrete frame. Table 51 gives safe loads for this construction. These are given only as a guide to approximate conditions, as many codes have governing stipulations.

Instead of using the terra cotta fillers, "Slagblock" is employed as an alternate for such construction, as well as in one-way ribbed slabs. These are 16″ square in plan and of varying depths, and are made of a slag concrete. The object of these blocks is not only to replace the terra cotta as a void producer, but tests have shown that they may be counted upon as equivalent poured concrete for shear and compressive values. The design of such a floor includes the blocks as a part of the effective concrete.† Lugs on the blocks are used to develop bonding action. An advantage claimed is that the bottoms of the blocks serve as T-action for the joists near the supports to supply compressive resistance.

Another type of two-way reinforced hollow tile floor is the Shuster system. This is a patented system which is similar to other types with special features.‡

129. "Natcoflor" Systems.

A patented system embodying the use of special terra cotta blocks and cement grout joists is called the "Natcoflor" tile system, as illustrated in Fig. 197. No concrete slab is used such as in the typical floor, and this system was developed to obtain the maximum sectional area of the tile where it will be of the most use in resisting the compressive stresses ordinarily taken by the concrete top slab. The dead weight of the floor itself is less than the typical terra cotta tile-joist floor, although the cost is about the same, but the saving effected is in the design of the supporting members. Floor depth, as well as dead load, is saved, as this system averages 2″ to 4″ less than other tile floors. An all tile ceiling for the plastering base is available, requiring no metal lath or a suspended ceiling. As the tile blocks join at the bottom, there is no danger of "joist lines" appearing in the ceiling later. Such construction may be combined with I-beams, as shown in the figure, or with concrete T-beams. Table 52 gives the essential features of the design.

130. The Use of Gypsum Tile or Cinder Concrete Blocks.

Gypsum tile are occasionally used as an alternate method of providing the voids in tile-joist construction, although they are not as commonly employed as steel tile or terra cotta blocks. The gypsum blocks provide a lighter filler than clay tile, offer a satisfactory plaster base, and good heat and sound insulation. The blocks are readily cut in the field to make adjustments. The standard width is 20″, and the joists are made 4″ to 6″ wide with a typical 2″ top slab. An objection to such filler blocks is that they require careful handling. Figure 198 (*a*) shows one form of construction, and (*b*) illustrates the details of the L-lock floor tile, manufactured by the Ellock Corporation, Buffalo, N. Y. Special tile ends or plaster boards are used to close the open ends of the tile and thus eliminate the waste of concrete. The design of such construction is similar to that of other one-way tile-joist systems. Cinder concrete blocks are now also used in a similar way.

* Patented by the Republic Fireproofing Co., New York City.

† This system, as well as others, has not as yet been accepted by all building departments. The designer, in any instance, should determine this point for a given locality.

‡ Data for this type of floor may be obtained from the Jno. T. McCoy Sales Corp., 54 Nassau St., New York City.

TABLE 51

SAFE LIVE LOADS FOR "REPUBLIC" 2-WAY FLOORS

Panel Size	40#			60#			75#			100#			120#		
	Simply Supp.	Partly Cont.	Fully Cont.	Simply Supp.	Partly Cont.	Fully Cont.	Simply Supp.	Partly Cont.	Fully Cont.	Simply Supp.	Partly Cont.	Fully Cont.	Simply Supp.	Partly Cont.	Fully Cont.
16'-0"×16'-0"	4+1½	4+1	4+1	4+2	4+1½	4+1	4+2	4+2	4+1½	4+2	4+2	4+2	4+2½	4+2	4+2
16'-0"×18'-0"	4+2	4+1½	4+2	4+2½	4+2	4+1½	5+2	5+2	4+2	4+2½	4+2	4+2	5+2½	4+2½	4+2½
16'-0"×21'-0"	6+2	4+2	4+2	6+2	6+2	4+2	6+2	6+2	4+2	6+2	6+2	4+2	6+2	6+2	6+2
18'-0"×18'-0"	5+2	4+2	4+1½	5+2	4+2	4+1½	5+2	5+2	4+2	5+2	4+2½	4+2	5+2½	5+2	4+2½
18'-0"×20'-0"	5+2	4+2	4+2	5+2	4+2½	4+2½	5+2½	5+2	4+2½	6+2	5+2½	6+2	6+2	5+2½	5+2
18'-0"×24'-0"	5+2	4+2½	4+2	5+2½	5+2	4+2½	6+2	5+2	5+2	6+2½	5+2½	5+2	6+2½	6+2	6+2
20'-0"×20'-0"	5+2	4+2½	4+2	5+2½	5+2	4+2	5+2½	5+2	4+2½	5+2½	5+2	4+2½	6+2½	5+2½	5+2
20'-0"×22'-0"	5+2½	4+2½	4+2	5+2½	5+2	4+2½	6+2	5+2½	6+2	6+2½	6+2	5+2½	7+2	6+2	5+2½
20'-0"×27'-0"	6+2	5+2	4+2½	6+2½	5+2½	5+2	6+2½	6+2	5+2½	7+2½	6+2½	6+2	8+2	7+2	6+2½
22'-0"×22'-0"	5+2½	5+2	4+2½	6+2	5+2	4+2½	6+2	5+2½	5+2	6+2	5+2½	5+2	7+2½	6+2½	6+2
22'-0"×25'-0"	6+2½	5+2½	5+2	7+2	6+2	5+2½	7+2	6+2½	6+2	7+2½	7+2	6+2½	8+2	8+2	7+2
22'-0"×29'-0"	7+2	7+2	6+2	7+2½	6+2½	6+2	8+2	7+2	6+2½	8+2½	7+2½	7+2	8+3	7+2	7+2
24'-0"×24'-0"	6+2½	5+2½	5+2	6+2½	5+2½	5+2	6+2½	6+2	5+2½	7+2	6+2½	6+2	8+2	7+2	6+2½
24'-0"×28'-0"	7+2	6+2	5+2½	7+2½	6+2½	6+2½	8+2	7+2	6+2½	8+2	7+2½	7+2	8+3	8+2	7+2
24'-0"×32'-0"	7+2½	6+2½	6+2	8+2	7+2	6+2½	8+2½	7+2½	7+2	10+2	8+2	7+2½			
26'-0"×26'-0"	7+2	6+2	5+2½	7+2	6+2½	6+2	7+2½	6+2½	6+2	8+2	7+2½	6+2½			
26'-0"×30'-0"	8+2	7+2	6+2½	8+2½	7+2½	7+2½	8+3	8+2	7+2½						
26'-0"×35'-0"	8+2	7+2	6+2½	7+2½	7+2½	7+2	10+2	8+2	7+2½						
28'-0"×28'-0"	8+2	7+2	6+2	8+2	7+2	6+2½									
28'-0"×32'-0"	8+2½	7+2½	7+2	10+2	8+2	7+2½									
28'-0"×37'-0"	10+2	8+2½	7+2½												
30'-0"×30'-0"	8+2½	7+2½	6+2½												
30'-0"×34'-0"	10+2	8+2½	8+2½												
30'-0"×40'-0"	10+2½	10+2	8+2½												

In the above table values, the first figure indicates depth of tile used and second, the concrete top.

Table values based on fibre stresses $\begin{cases} 750 \text{ lbs. per sq. in., compression on concrete.} \\ 18{,}000 \text{ lbs. per sq. in., tension on steel.} \end{cases}$

For allowable stresses of 650# & 16,000# on concrete and steel respectively add ½" to depths shown above.

An allowance of 25 lbs. per sq. ft. has been made in the above table values, for cinder fill, finished flooring and plastered ceiling.

TABLE 52

"NATCOFLOR" LONG-SPAN SYSTEM

Data for Designing Purposes

The upper figures in table denote the depth of tile; the lower figures indicate the area of reinforcing steel required in each mortar rib. Thickness of floor = depth of tile.

The table below is so arranged that it may be used for floor slabs freely supported at both ends, semi-continuous, or continuous.

For slabs freely supported at both ends (simple span), use loads given opposite $\frac{W \cdot L}{8}$.

For slabs freely supported at one end and continuous over other support, use loads given opposite $\frac{W \cdot L}{10}$.

For slabs continuous over both supports, use loads given opposite $\frac{W \cdot L}{12}$.

For semi-continuous and continuous spans proper reinforcement must be provided in top of slab over support to take care of negative bending moment.
Shearing stress must be investigated in all cases.
Safe compression. 1200 lb./□" on net section. Mortar 1 cement : 2½ sand.

No Concrete Top

f_m & f_t = 1000 lb. per sq. in.
f_s = 16,000 lb. per sq. in.

$\frac{Et}{Es} = \frac{1}{10}$

¾-in. below reinforcement
2-in. mortar ribs, 13-in. on centers

Bending Moment	Total Safe Loads (Dead & Live)											
$\frac{W \cdot L}{12}$	150	165	180	195	210	225	240	260	300	335	375	450
$\frac{W \cdot L}{10}$	125	135	150	160	175	185	200	220	250	280	310	375
$\frac{W \cdot L}{8}$	100	110	120	130	140	150	160	175	200	225	250	300
Span 6'-0"	4/.16	4/.17	4/.19	4/.20	4/.22	4/.23	4/.25	4/.28	4/.31	4/.35	4/.39	5/.35
" 7'-0"	4/.21	4/.23	4/.26	4/.27	4/.30	4/.32	4/.34	4/.38	5/.32	5/.36	5/.40	5/.48
" 8'-0"	4/.28	4/.30	4/.33	4/.36	4/.39	5/.31	5/.33	5/.37	5/.42	5/.47	5/.52	6/.49
" 9'-0"	4/.35	4/.38	5/.32	5/.34	5/.37	5/.39	5/.42	5/.46	5/.53	6/.46	6/.51	6/.62
" 10'-0"	5/.33	5/.35	5/.39	5/.42	5/.46	5/.48	5/.52	6/.45	6/.51	6/.57	6/.63	7/.63
" 11'-0"	5/.39	5/.43	5/.47	5/.50	6/.43	6/.46	6/.49	6/.54	6/.62	7/.57	7/.63	8/.65
" 12'-0"	5/.47	5/.51	6/.44	6/.47	6/.51	6/.54	6/.59	6/.65	7/.61	7/.68	7/.75	8/.77
" 13'-0"	6/.43	6/.47	6/.52	6/.55	6/.60	6/.64	7/.57	7/.63	7/.71	8/.67	8/.75	9/.79
" 14'-0"	6/.50	6/.54	6/.60	6/.64	7/.58	7/.61	7/.66	7/.73	8/.70	8/.78	9/.75	9/.91
" 15'-0"	6/.57	6/.62	7/.57	7/.61	7/.67	7/.70	7/.76	8/.70	8/.80	9/.78	9/.86	10/.93
" 16'-0"	6/.65	7/.58	7/.65	7/.69	7/.76	8/.67	8/.73	8/.80	9/.79	9/.89	10/.87	12/.86
" 17'-0"	7/.61	7/.66	7/.73	8/.66	8/.72	8/.76	8/.82	9/.79	9/.90	10/.89	10/.99	12/.97
" 18'-0"	7/.68	7/.74	8/.69	8/.74	8/.81	9/.74	9/.80	9/.88	10/.89	10/1.00	12/.90	12/1.08
" 19'-0"	7/.76	8/.69	8/.77	8/.82	9/.78	9/.83	9/.89	10/.88	10/.99	12/.90	12/1.00	
" 20'-0"	8/.71	8/.77	8/.85	9/.79	9/.87	9/.92	10/.88	10/.97	12/.89	12/1.00	12/1.10	
" 21'-0"	8/.78	8/.85	9/.82	9/.88	10/.85	10/.90	10/.97	12/.87	12/.98	12/1.10		
" 22'-0"	9/.75	9/.81	9/.90	10/.85	10/.93	10/.99	12/.86	12/.95	12/1.08			
" 23'-0"	9/.82	9/.89	10/.87	10/.93	12/.83	12/.87	12/.94	12/1.04	12/1.18			
" 24'-0"	9/.89	10/.86	10/.95	12/.82	12/.90	12/.95	12/1.03	12/1.13				
" 25'-0"	10/.86	10/.93	12/.84	12/.89	12/.97	12/1.03	12/1.11					
" 26'-0"	10/.93	12/.81	12/.90	12/.96	12/1.05	12/1.12						
" 27'-0"	12/.81	12/.88	12/.97	12/1.04	12/1.13							
" 28'-0"	12/.87	12/.94	12/1.05	12/1.12								
" 29'-0"	12/.94	12/1.01	12/1.12	12/1.20								
" 30'-0"	12/1.00	12/1.08	12/1.20									

Design Data

Size	Cu. ft. Mortar per sq. ft.	Weight in lb. Each Tile	Weight in lb. Floor sq. ft.	Size	Cu. ft. Mortar per sq. ft.	Weight in lb. Each Tile	Weight in lb. Floor sq. ft.
4"	.051	23	28	8"	.102	34	45
5"	.064	28	34	9"	.115	37	48
6"	.077	30	39	10"	.128	40	52
7"	.090	32	42	12"	.154	47	59

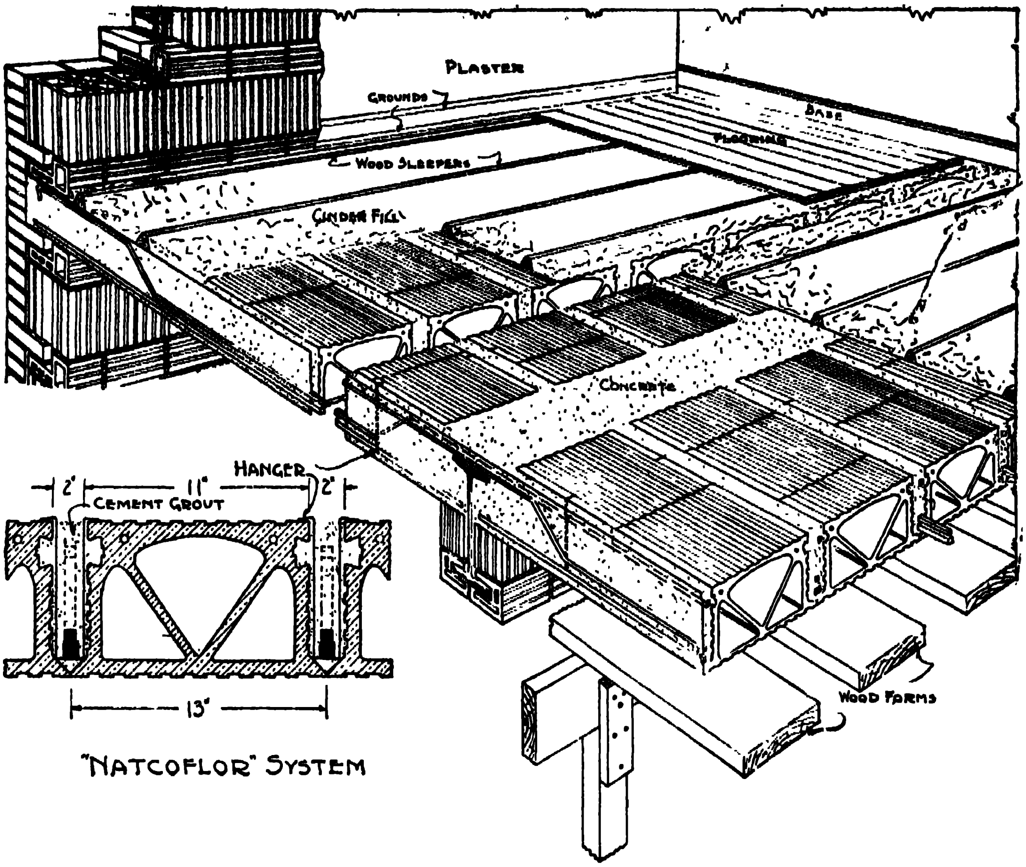


Fig. 197

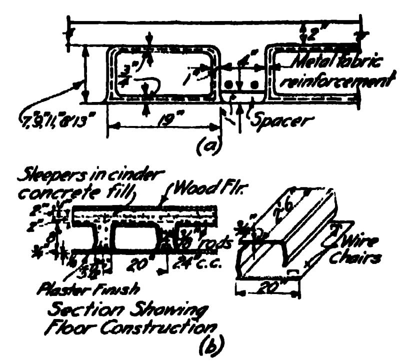


Fig. 198

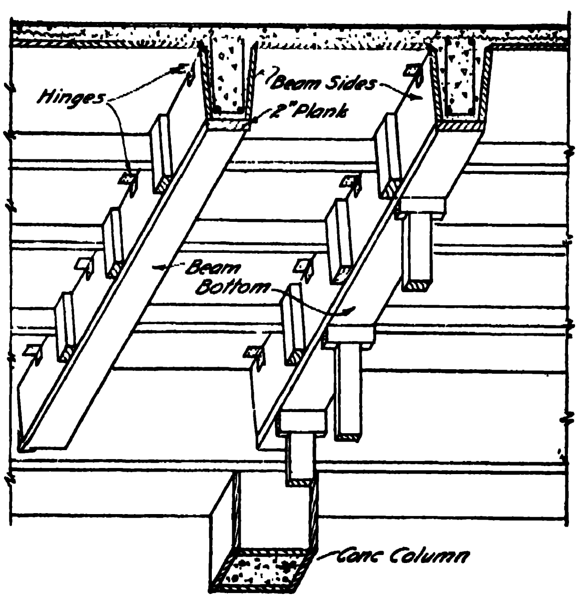
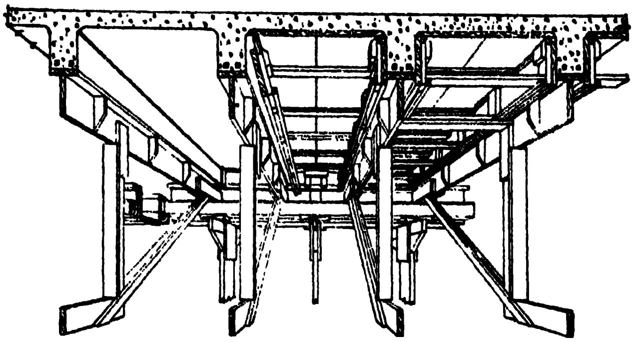


Fig. 199

131. The Use of Wooden Boxes.

Another method of constructing the voids in concrete joist construction is illustrated in Fig. 199. Inverted wooden boxes covered with sheet metal, and hinged so that they may be collapsed have been used successfully to form the ribbed slab construction.* Figure 200 also shows an alternate scheme, using wood removable forms. These allow re-use a great number of times, and since they are removed, the dead weight of the floor compares favorably with other forms of construction. Another advantage is that more convenient and rigid attachment of inserts, electric outlets and sleeves is possible than in the other types. The design of such framing involves only the principles heretofore discussed.

* This system was used at the University of Wisconsin. For a more complete discussion, see Hool's "Reinforced Concrete Construction," Vol. II. — McGraw-Hill Book Co., Inc.



REMOVABLE WOOD FORMS

FIG. 200

Courtesy of National Fireproofing Co.

PLATE 15 VIEW OF TERRA COTTA ARCH CONSTRUCTION

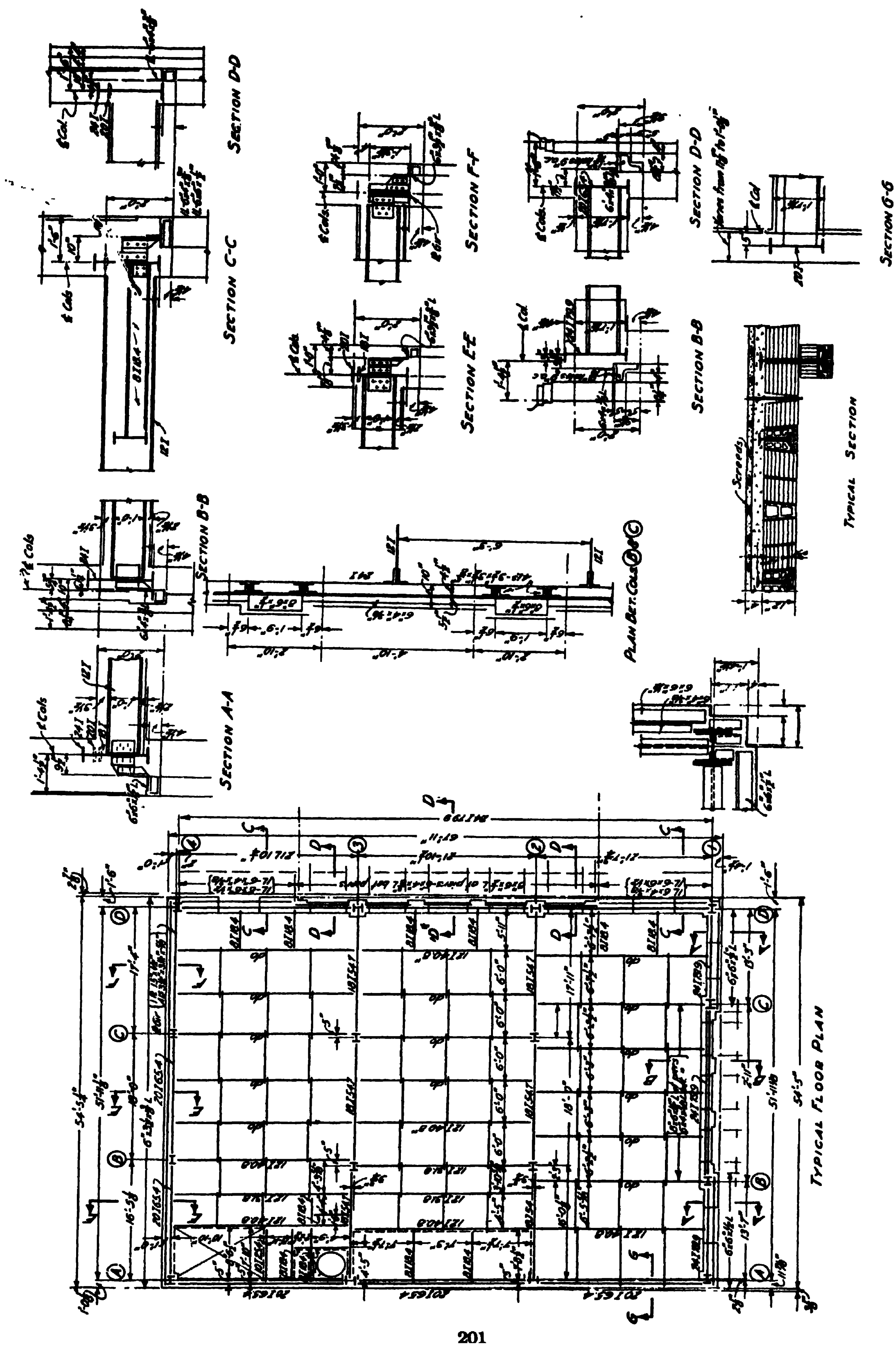


PLATE 16 TYPICAL FRAMING PLAN
TERRA COTTA ARCH CONSTRUCTION

CHAPTER 9

FLOOR ARCH CONSTRUCTION

132. General Considerations.

One of the methods used in constructing fire-resisting floors is that of arches sprung between rolled steel floor beams. In general, the materials used in the arches must be such that they will offer a real resistance to fire and the transmission of heat without undue disintegration, and to the action of the water from hose streams. The materials should make a floor which is as nearly watertight as possible. Terra cotta, concrete, and brick are the materials which have been used for such work, but in present practice, terra cotta is more commonly used, concrete occasionally, and brick seldom, for this type of floor. Some of the advantages of terra cotta arch floor systems are:

(1) Fire resistance of the arch, and ample protection for the steel beams,

(2) Relatively light dead loads and a permanent and sound-proof floor,

(3) Ample strength for all ordinary loads,

(4) Less interference with the operations in other parts of the job, and speed of erection.

Arch construction is not particularly adaptable to irregular framing, and the supporting beams should be parallel to receive the thrusts from the arches in the proper manner. Irregular framing also requires awkward details for the beam haunches.

Section 9A

TERRA COTTA ARCHES

133. Types of Arches.

The various kinds of terra cotta floor arches which are used may be classed as flat, segmental and reinforced. Figure 201 illustrates the first two types. The third is discussed in Art. 137. The general construction is the same in each case except for the variation in the arch itself. The typical wood floor with screeds may be used, or a granolithic, tile or marble finish floor may be employed. The arch blocks adjacent to the steel beams are so constructed that they project below the soffits of the beams, and with the addition of a small soffit block, fire protection is afforded. The space between the top of the arch and the flooring is filled with cinder concrete, which protects the top flanges of the beams and provides a space in which pipes, conduits, screeds and so on, may be placed.

Each type of arch has its own particular advantages, and the choice depends upon the kind of building, local conditions, and the magnitude of the loads to be carried. If a flat arch is used, the ceiling may be applied directly to the tile, whereas a segmental arch requires a suspended ceiling, if a flat ceiling is required. A flat ceiling is a distinct advantage, as it reflects light and deflects heat more advantageously. Segmental arches, being built to the form of a circular segment, are stronger than flat arches of the same depths and spans. For a given floor load, therefore, a segmental arch requires less depth of arch and is consequently more economical of terra cotta. However, a higher cost of setting, of falsework and of ceiling construction results. To serve economy, any arch should theoretically develop the full strength of the floor beams, and if a flat arch is properly designed, this may be done, but in the case of a segmental arch, this is not always easy to accomplish. In view of the above, a flat arch lends itself more readily to average conditions and is by far the more commonly used. Segmental arches are adaptable where heavy floor loads are encountered and where a flat ceiling is not required, as in warehouses, lofts, sidewalk construction and factories. They are also occasionally used for roof construction. An advantage which deserves consideration is that longer spans may be used, and in many cases intermediate floor beams may be eliminated. Disadvantages are that it is difficult to fit the tile around tie-rods, and that more top filling is required. Reinforced terra cotta floor arches are principally confined to patented forms of construction, as discussed in Art. 137.

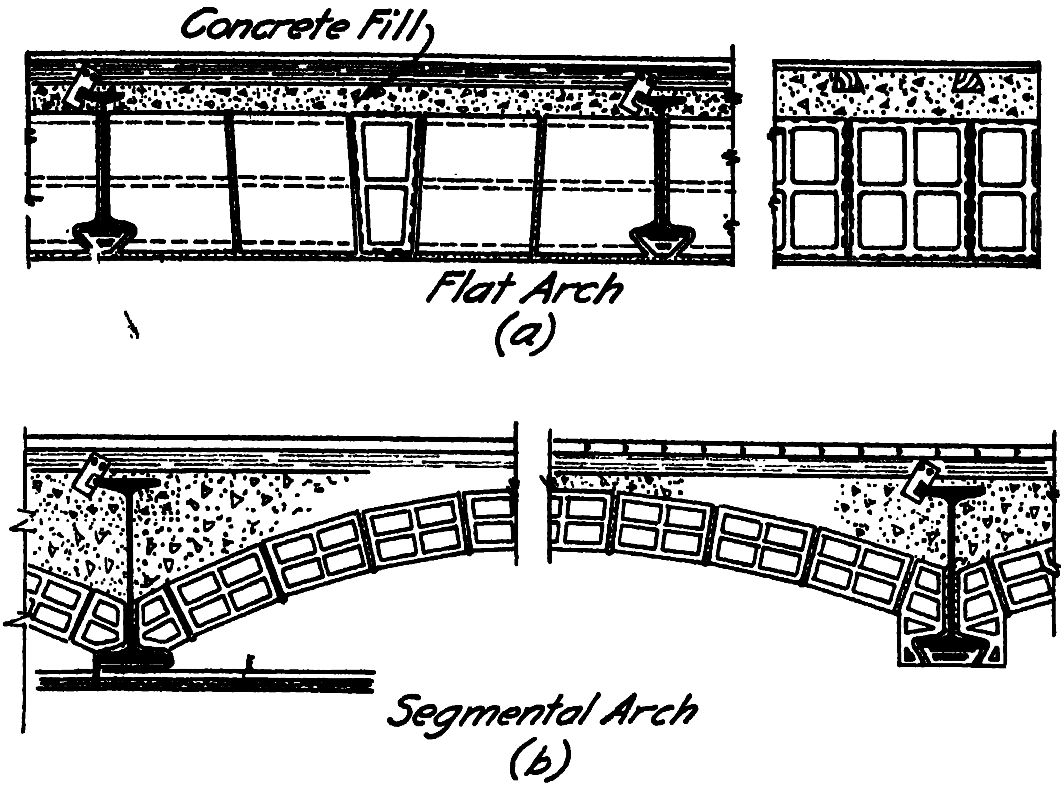


Fig. 201

134. General Theory of Floor Arch Construction.

The nature of floor arch construction is such that a lateral thrust is exerted on the floor beams. In a flat arch, the faces of the blocks are tapered or " voussoired " and the central key block is wedged in, so that when a load is supported by the floor, a thrust is developed by the action of the blocks. A segmental arch, since it is really built in the form of a true arch, exerts a natural thrust. For any particular floor beam, it is necessary to determine the amount of load which causes a thrust. Thus for an intermediate beam, the thrusts caused by the dead load on each side balance each other. Consequently, the maximum condition which may occur is to consider the live load on one side only for an intermediate beam. For beams at the outside edges of end panels, or those surrounding openings in the floor, the thrust due to both dead and live loads must be considered, as there is no thrust from the opposite side to balance it.

The thrust per linear foot of beam, p, may be obtained by considering a transverse strip of the arch, 1′-0″ wide, as a simple beam. The maximum bending moment in a simple beam may be expressed by $\frac{w \cdot L^2}{8}$ ft.-lbs. In Fig. 202, the span of the arch in feet is designated by L_a. If w_0 represents the effective load on the arch in #/□′ in producing thrust (the L.L. for an intermediate beam and the L.L. + D.L. for an outside beam), it is also the load per linear foot of arch for a 1′-0″ strip. The bending moment may then be expressed by $M = \frac{w_0 \cdot L_a^2}{8}$

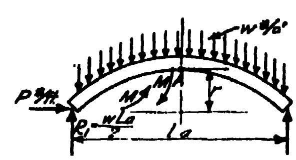


Fig. 202

for this case. This must be resisted by the reaction to the thrust, acting with a lever arm equal to the effective rise of the arch, as illustrated in Fig. 202. Taking moments about A,

$$\Sigma M_A = \frac{w_0 \cdot L_a}{2}\left(\frac{L_a}{2}\right) - \frac{w_0 \cdot L_a}{2}\left(\frac{L_a}{4}\right) - \frac{p \cdot r}{12},$$

in which r represents the effective rise of the arch in inches. Solving

$$\frac{p \cdot r}{12} = \frac{w_0 \cdot L_a^2}{8} \text{ (as stated above), or}$$

$$p = \frac{3\,w_0 \cdot L_a^2}{2\,r} \qquad (S\text{-}39)$$

(The value p is the thrust per linear foot of beam in pounds.)

Tie rods are introduced into the floor system, as illustrated in Fig. 203, to take the thrusts exerted.

This is a very important feature, as stiffness is added to the frame. The size is almost invariably made $\frac{3}{4}''\phi$ in order to conform to the punching of the other holes in beams, which is commonly $\frac{13}{16}''$.* The tensile strength of a rod, if threaded, as is the case here, is based upon the net section. For a $\frac{3}{4}''$ rod, this

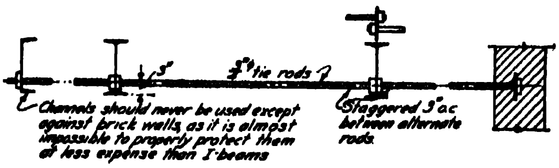


FIG. 203

is 0.30□", and using the usual allowable stress, the tensile strength, T_r, is $0.30 \times 16{,}000 = 4800\#$. The theoretical spacing of tie rods in feet, L_s, may be calculated by dividing the strength of a rod by the thrust per foot of beam, or

$$L_s = \frac{T_r}{p}. \qquad (S\text{-}40)$$

For $\frac{3}{4}''$ rods,
$$L_s = \frac{4800}{p}.$$

In the usual case, the spacing so calculated will not always be a reasonable value, and since the functioning of tie rods is more or less nominal, rules of thumb are usually employed in practice. Steel beams should be laterally supported at distances equalling 20 flange widths unless the working stress is reduced (Art. 179) in any case, so that a value sometimes stipulated as a maximum spacing is 15 flange widths of the floor beam.

SPECIFICATION CLAUSE†

Beams and channels acting as skewbacks for arches shall be designed to resist the lateral thrusts in addition to their vertical loads, and the tie rods, not less than three-fourths of an inch in diameter, shall be placed as near the line of thrust as practicable, and in any event shall be spaced not more than eight times the depth of the beams, and not more than eight feet.

Since depths and flange widths of beams vary, a more feasible rule is to use a maximum spacing of 6'-0", and to space the tie rods at equal distances across the length of the beam.‡ As the stresses due to bending are ordinarily a maximum near the point of mid-span, the spacing could theoretically be increased toward the ends of the beam, but a uniform spacing is used for simplicity. In no case should the spacing exceed the value obtained from the formula (S-40). Rods must not be spaced, "hit-or-miss," as they bear a definite structural relation to the cross jointing of the arch blocks. They are embedded in the joint and thus should be at such intervals as will conform to the joints of the tile,—hence they should be in even foot spacings, as the blocks are 12" wide. For purposes of stiffening a floor frame, it is desirable to have the tie rods in a continuous line across the floor. This also protects against unforeseen thrusts or excess loads. For this reason, it is quite common in practice to apportion the tie rods for the outside panels and to make those in the interior panels the same. Theoretically the tie rods should be placed in the line of the thrust. At least they should be placed near the bottom flanges of the beams and still have at least 2" fire protection. Accordingly, tie rods are commonly 3" from the bottoms of the beams and staggered back and forth across a building 3" o.c., so that the nuts may be turned without interference (Fig. 203).

The steel supporting beams are subjected to bending moments caused by the forces acting in two directions, namely, the vertical load and the thrust. The bending moment in a vertical direction may be calculated in the usual manner, or

$$M_{1\text{-}1} = \frac{w \cdot L^2}{8} \text{ ft.-lbs, in which}$$

w = the total load per linear foot on the beam, and
L = the span of the beam in feet.

The tie rods tend to fix the beam at their points of connection, so that it acts more or less as a continuous beam in its lateral bending. For fully continuous beams, the moment is expressed by $M = \frac{w \cdot L^2}{12}$. In this particular case, the span is the spacing of tie rods, L_s, and the load per foot is the thrust, p. Hence the lateral bending moment is

$$M_{2\text{-}2} = \frac{p \cdot L_s^2}{12} \text{ ft.-lbs.}$$

Figure 204 (*a*) illustrates the action. In a simple beam, the maximum compression occurs at the top fibres and the maximum tension at the bottom fibres. For lateral bending, the maximum compression occurs on the side toward the thrust, and the maximum tension on the side away from the thrust. These respective stresses may be designated by $s_{1\text{-}1}$ and $s_{2\text{-}2}$. The arches do not exert a uniform pressure throughout their surfaces of contact with the sustaining beams on account of the varying friction and bond of the component parts. Theoretically, the maximum stress cannot be obtained

* Rods $\frac{5}{8}''$ and $\frac{7}{8}''$ in diameter are occasionally used, but these sizes should be avoided in order to avoid complicating the punching in the beam webs.

† The Building Law of the City of Boston.

‡ An interesting article, "Tie Rods for Floor Arches," by Mr. R. Fleming, may be found in the Engineering News Record, March 18, 1915.

exactly by adding the stresses due to vertical and lateral bending,* but such a procedure is considered to be exact enough for commercial design when conservative working stresses are used. On this basis,

$$s_{1-1} = \frac{M_{1-1}}{\frac{I}{c}(1-1)} \quad \text{and} \quad s_{2-2} = \frac{M_{2-2}}{\frac{I}{c}(2-2)} \quad \text{or}$$

$$s_{max} = s_{1-1} + s_{2-2}.$$

This is illustrated in Fig. 204 (*b*).

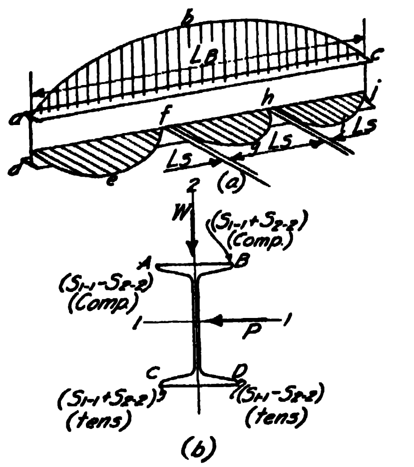


FIG. 204

The maximum allowable combined stress is often specified as 18,000#/□″. This value is higher than the customary 16,000#/□″. In order to eliminate some of the " cut and try " which may occur in such design, a beam may be selected for the vertical section modulus required in the usual way, and then the next larger size tested in the light of the foregoing discussion.

The strength of the arch itself is limited by the safe compressive strength of the terra cotta and its mortar joints. There are three different compositions of structural terra cotta, namely, dense (hard burned), semi-porous, and porous. The semi-porous tile is used principally for floor arches because the porous material, although possessing the best fire resistance, has a low compressive strength. The dense tile, while stronger than the semi-porous, is less fire-resisting, and is not as commonly used for flat-arch construction.

SPECIFICATION CLAUSE†

Hollow terra cotta tile used for floor or roof arches shall be hard burned or semi-porous and of uniform density and hardness. All terra cotta arches shall be properly keyed. The key blocks shall always be placed within the middle third of the span.

The ultimate strength of terra cotta varies from 2500#/□″ to 4000#/□″, depending upon the grade, but the various shaped blocks seem to have about the same strength for a given sectional area. When blocks are placed so that they take thrust across the webs and walls, these strengths are quite materially reduced. The factor of safety is usually taken from 5 to 7 according to the conditions surrounding the design.

SPECIFICATION CLAUSES†

Terra cotta floor tile, when tested on end and faced with Portland cement, shall give an average compressive strength of not less than 2500 lbs. per square inch of net area. The average strength shall be computed from the results of tests of ten average tile.

The allowable extreme fibre stress in compression in terra cotta floor tile shall be taken as 500 lbs. per square inch on net section.

The sectional area of a tile arch is determined by a plane at right angles to all webs and of a 1′-0″ width parallel to the beams. The following table gives the common sectional areas.

TABLE 53

CROSS-SECTIONAL AREAS OF TERRA COTTA

Depth (Inches)	Areas (Sq. Ins.)	
	Flat	Segmental
4″	—	28
6	31	36
7	34	—
8	37	43
9	40	—
10	43	47
12	49	—
15	58	—

Illustrative Prob. 134a. What is the theoretical depth of the segmental arch required for the following data?

L.L. = 300#/□′ Span of arch = 16′-0″

Effective rise = 20″

$$p = \frac{3\,w_0 \cdot L_a^2}{2\,r} = \frac{3 \times 300 \times (16)^2}{2 \times 20} = 5760\#/\text{ft.}$$

$\frac{5760}{500}$ 11.6□″ theoretically required.

A 4″ arch is theoretically satisfactory

In practice, the strength of the arch is very seldom the governing feature, and the arch is generally made thicker than theoretically required. The cost of the fill, which is used over the tile, is greater than that of the terra cotta, and also the dead load is

* For a more theoretical discussion, refer to Proc. A.S.C.E. on $\frac{I}{c}$ polygon data.

† National Board of Fire Underwriters.

increased more by using a thin arch and a deeper fill (see Fig. 205). Furthermore, the question of the fire protection of the steel beams is essential. This is usually provided by using the same material as that of the arch.

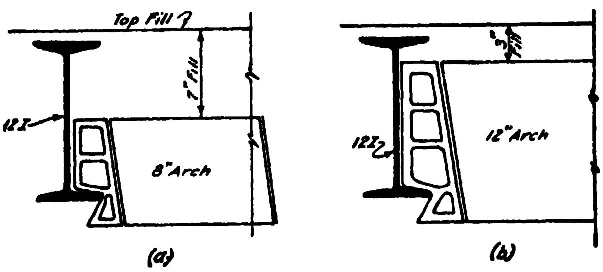


FIG. 205. RELATION OF TILE AND BEAM DEPTHS

(a) for economy of tile
(b) for reduced fill and greater protection

SPECIFICATION CLAUSE*

If hollow terra cotta tile be used for protection, the lower flanges of beams and similar members shall be encased either by lugs which form part of the skewbacks and extend around the flanges meeting at the middle; or by tile slabs held in position by dove-tailed lugs projecting from the skewbacks. In either case care shall be taken to insure that all joints be solidly filled with mortar.

The weight of the protecting materials should be included in that which the beam is designed to carry. Figure 206 illustrates the typical methods of providing fire protection to various beam and girder sections. The question of the weight of beam-protecting tile is included as a rule in the arch weight. When girders are to be protected as in Fig. 206, the following illustrative example for determining the **haunch weight** is typical.

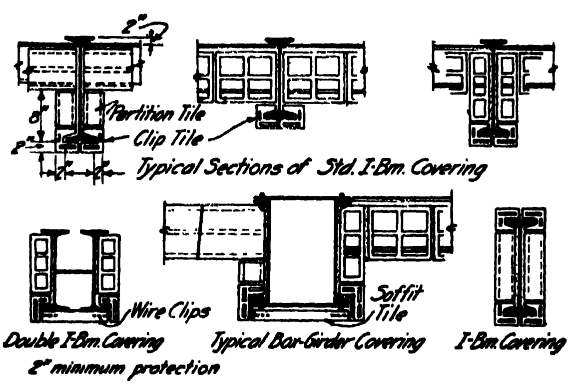


FIG. 206

Illustrative Prob. 134b. Tabulate the uniform load per foot for the girder and its fire protection material in Fig. 206, if the girder is a 24 I 79.9 and the floor beams 12 I 31.8. Depth below arch blocks to soffit of girder = 24 − 2 − 12 − 2 = 8″

* National Board of Fire Underwriters.

8″ + 2″ F.P. = 10″
Flange width = 6″
6″ + 2″ + 2″ = 10″ width
Area of Section of Haunch = 10 × 10 = 100□″
Net Section (60%) = 60□″
60 × 12 × 0.06#/cu. in. = 43#
Wt. of Girder = 80
Girder and haunch = 123#/ft.

Prob. 134c. What is the weight of beam and haunch per linear foot for a 20 I 65.4 girder if the floor beams are 10 I 25.4?

135. Flat Terra Cotta Arches.

A flat terra cotta arch may be built in various ways, such as side construction, end construction, or a combination of the two (Fig. 207). These names are assigned according to the location of the faces of the tile with respect to the thrust. In side construction, the cells run parallel to the beams, and in end construction, at right angles to them. Side construction offers the advantage that when a single tile is broken or removed, the strength of the arch is not impaired, as the blocks break joints. Hence when holes are cut through the floor no great damage is done. However, such construction is seldom used, as it is not the natural way to place the tile for effective resistance to thrust. Figure 208 illustrates the typical end construction. Such an arch is stronger than one of side construction, as the webs of the end blocks obtain a direct bearing against the webs of the beams and the thrust is transmitted directly through the webs and walls of the blocks. The key block is in side bearing, but it is relatively thin and stiff. Some of the disadvantages of such construction are:

(1) The fire protection of the beams is not complete.

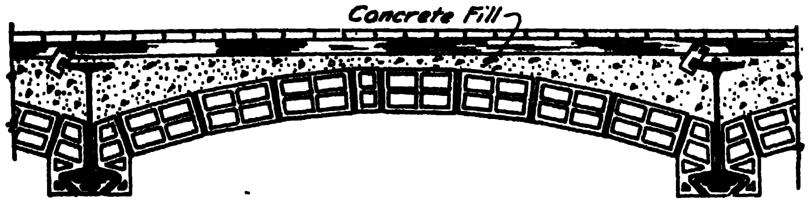
(2) It is difficult to bed the skewbacks.

(3) Close inspection and more perfect tile are required, as the ribs must be opposite each other to be effective.

(4) The edges of the end blocks are easily broken in transportation and in erection, thereby impairing the bearing areas.

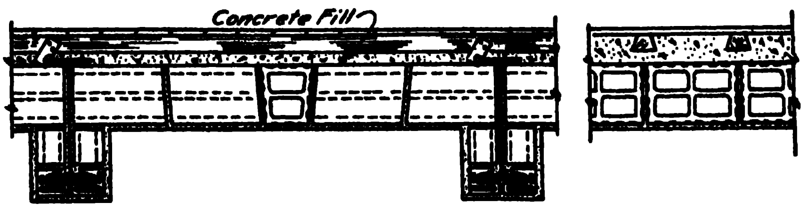
(5) If a single block is removed or broken, the strength is impaired to a greater degree than in side construction, and the remaining blocks are left to depend upon the mortar bond alone.

In order to overcome the objections to the two types previously discussed, a combination of side and end construction is the most commonly used, as shown in Fig. 209. This has the typical key at the center in side bearing and a side construction skew block is used at each beam. This provides better fire protection to the beam because of the continuous tile wall parallel to it. Such construction is



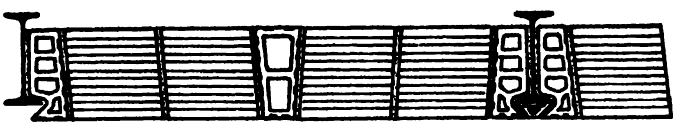
Standard Segmental Arch

(a)



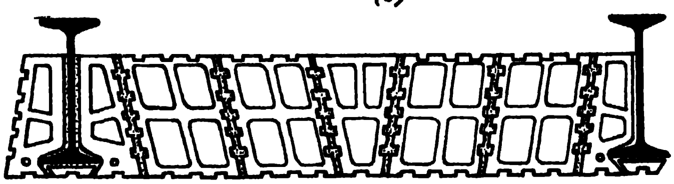
Raised on Floor Beams for a Paneled Ceiling effect

(b)



Combination of End and Side Construction Flat Arch

(c)



Hollow tile flat arch - side construction

(d)

FIG. 207. TERRA COTTA FLOOR ARCHES

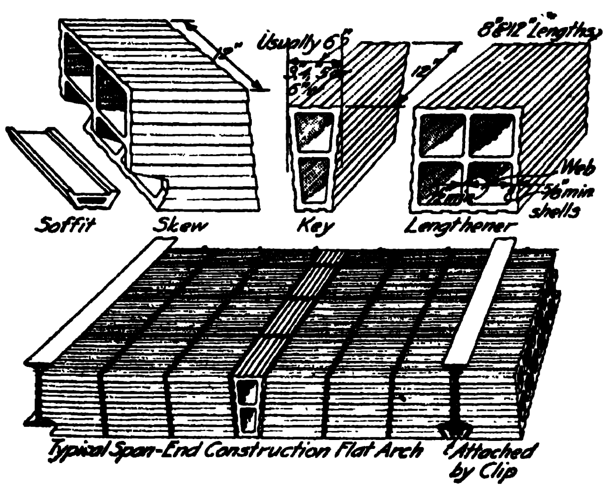


FIG. 208

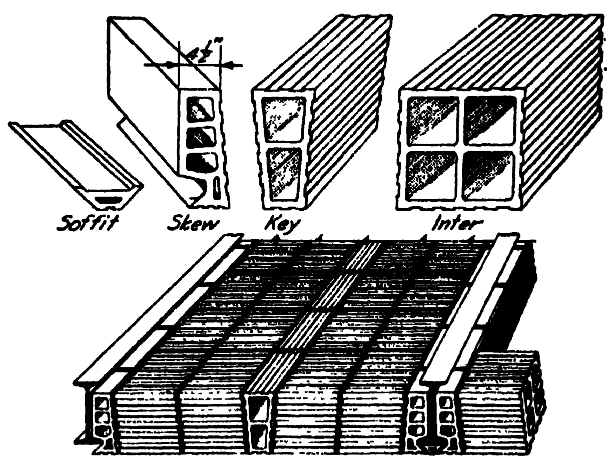


FIG. 209

commonly called "end construction," and, in fact, it virtually is, as the lengtheners which supply the bulk of the arch are in end bearing. It is the type almost invariably used. Figure 210 shows some typical arch blocks.

Steel beams should be placed on or reasonably close to all column center-lines, and intermediate beams should divide the panels into equal spaces when possible. It generally is more economical of steel if the beams are of the minimum weight and spaced as far apart as their strength will permit.

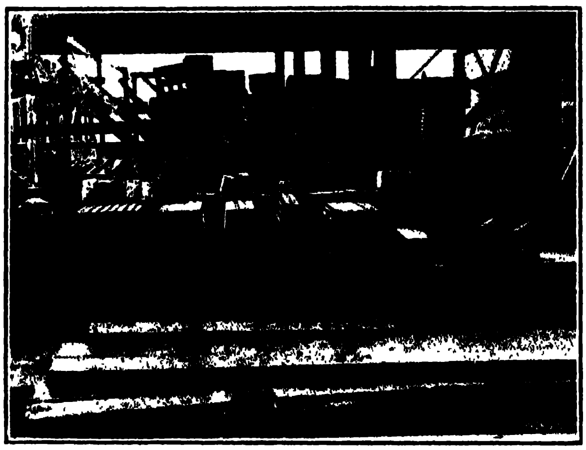


Fig. 210

On the other hand, if the span of the arch is too great, there is a tendency of failure at the haunch, and a large thrust is exerted upon the beams. Furthermore, the question of expansion is important. The coefficient of expansion of terra cotta is about twice that of steel, and is not uniform. If too great an expanse of arch occurs, the cumulative deformation caused by expansion may cause the terra cotta to spall at the beams and leave them exposed. Consequently, many specifications limit the maximum spacing of floor beams to 8′-0″ o.c.

Advantages of this type of construction are that good speed of erection is possible, that fewer cold weather precautions are necessary, and that a panel can be changed quite easily without affecting the whole floor. Disadvantages are the effects of expansion, and the lack of ideal protection for the steel against fire as well as corrosion.

The depth of the arch is controlled by its span and the load to be carried. The compression caused by the thrust is within safe limits in most cases (Illustrative Prob. 134*a*). Other considerations besides the economy of tile are influential, such as the stiffness of the floor, its fire-resistance, the cost and the dead weight of the fill, and the fire protection of the beams. For these reasons, the usual code stipulates the minimum thickness of arch allowable.

SPECIFICATION CLAUSE*

Flat arches shall have a depth of not less than 1¾ inches for each foot of span between the beams, this not to include any portion of the depth of tile that projects below the under side of the beams. The total depth shall in no case be less than 9 inches, and the tile shall have not less than three cellular spaces in the depth.

Table 54 gives the total safe loads per square foot that a given arch can safely sustain. **In practice, the designer commonly uses a depth of tile equal to that of the intermediate floor beam,** so that the latter will be concealed. The terra cotta is commonly set 1½″ below the tops of the beams so that the soffit block lines up with the bottom of the other blocks, as illustrated in Fig. 209.

TABLE 54

SAFE LOADS PER SQ. FT. FOR FLAT T.C. ARCHES

(Dead and Live)

Factor of Safety of 7

Arches	6 in.	7 in.	8 in.	9 in.	10 in.	12 in	15 in.
Net Sectional Areas	27 sq. in.	27 sq. in.	27 sq. in.	27 sq. in.	36 sq. in.	36 sq. in.	36 sq. in.
Average Wt. per Sq. Ft.	26 lb.	30 lb.	32 lb	36 lb.	40 lb.	48 lb.	56 lb.
Span — Ft. and In.	lb.	lb	lb.	lb.	lb.	lb.	lb.
3-0	420	490	560	630	933	1120	1400
3-3	357	417	477	537	795	954	1193
3-6	308	360	411	462	685	823	1028
3-9	268	313	358	403	597	716	895
4-0	236	276	315	354	525	630	786
4-3	...	244	279	314	465	558	697
4-6	...	218	249	279	415	497	622
4-9	...	...	223	251	372	447	558
5-0	...	...	201	227	336	402	504
5-3	...	...	182	205	305	365	457
5-6	...	...	...	187	277	333	417
5-9	...	...	...	171	254	305	381
6-0	...	...	...	157	233	280	350
6-3	...	...	...	...	214	258	322
6-6	...	...	...	...	198	238	298
6-9	...	...	...	...	...	221	276
7-0	...	...	...	...	...	206	257
7-6	...	...	...	...	...	178	223
8-0	...	...	...	...	...	157	197
8-6	...	...	...	...	...	...	174
9-0	...	...	...	...	...	...	155
9-6	...	...	...	...	...	...	140
10-0	...	...	...	...	...	...	126

The determination of the dead load to be carried is a matter of estimating in advance of the design, as in any case. If the construction described in the previous paragraph is used, the weight of the cinder fill may be based upon a 3½″ thickness, as a thickness of 2″ is commonly provided over the

* National Board of Fire Underwriters. Other codes are similar. The N. Y. Code specifies that the depth of the arch in inches shall be the length in feet times 1½ plus the thickness of the fireproofing below the beams.

tops of the beams. The weights of the finish flooring may be obtained from Table 30. The weight of the tile may be estimated from Table 54. The plastering for the ceiling is usually applied directly to the terra cotta and no lath is required as the tile are scored, so that 5#/□′ is a fair estimate for such work.

The zones of greatest pressure in a flat arch are near the top of the key and the bottom of the skewbacks, as indicated in Fig. 211. For lack of more positive information, an empirical rule is used to establish the value of the effective rise, namely to be 2.4″ less than the depth of the arch.* This has been derived by direct arch tests.

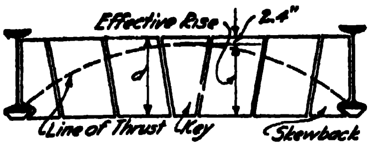


FIG. 211

With the above data established, the thrust, as described in Art. 134, may be calculated from

$$p = \frac{3\,w_0 \cdot L_a^2}{2\,r}.$$

The spacing of tie rods may also be fixed, the values of $M_{1\text{-}1}$ and $M_{2\text{-}2}$ calculated (Art. 134), and the required size of the beams found.

Illustrative Prob. 135a. Design a typical interior panel 20′-0″ × 20′-0″ of flat terra cotta arch construction. L.L. = 100#/□′. See Fig. 212.

L.L.	= 100	Try 5′-0″ spacing of beams.
Fin. Flr.	= 3	12″ Arch amply strong
Sub Flr.	= 3	(Table 54).
3½″ Cinder Fill	= 28	
12″ Arch	= 48	
Steel Beams	= 7	
Pl. Ceiling	= 5	
T.L.	= 194 say 195#/□′	

Vertical load per foot on beams

$5 \times 195 = 975\#/\text{ft.}$

$M = 1.5\,w \cdot L^2 = 1.5 \times 975 \times (20)^2 = 584{,}000''\#$

$\frac{I}{c}\,(\text{trial}) = \frac{584{,}000}{16{,}000} = 36.4''^3$ Try 12 I 40.8

$\text{Rise} = 12 - 2.4 = 9.6''$

$p = \frac{3\,w_0 \cdot L_a^2}{2\,r} = \frac{3 \times 195 \times (5)^2}{2 \times 9.6} = 762\#/\text{ft.}$†

$L_s = \frac{4800}{p} = \frac{4800}{762} = 6.3'$ *Use* $\frac{3}{4}''\ \phi$ *Tie Rods* 5′-0″ *o.c.*†

$M_{1\text{-}1} = 466{,}000''\#$ (from above)

$M_{2\text{-}2} = \frac{p \cdot L_s^2}{12} = \frac{762 \times (5)^2}{12} = 1590'\# = 19{,}070''\#$

$s_{1\text{-}1} = \frac{584{,}000}{44.8} = 13{,}020\#/\square''$

$s_{2\text{-}2} = \frac{19{,}070}{5.3} = \underline{3{,}600}$

$16{,}620\#/\square''$ O.K.

Use 12″ T.C. arch
12 I 40.8 beams.

Floor beam concentration on girder

$$975 \times 20 = 19{,}500\#$$

Assume wt. of girder and haunch = 125#/ft. (Illustrative Prob. 134b.)

$$R_1 = \frac{3}{2} \times 19{,}500 + \frac{125 \times 20}{2} = 30{,}500\#$$

$$M_{max} = 30{,}500 \times 10 - 19{,}500 \times 5 - \frac{125 \times (10)^2}{2} = 201{,}200'\#$$

$$\frac{I}{c} = \frac{201{,}200 \times 12}{16{,}000} = 150.6''^3$$ *Use 24 I 79.9 Girders.*

Figure 212 shows a typical engineer's sketch. Figure 213 shows a portion of the structural frame erected, and Fig. 214 shows the centering supported by the beams in a similar panel, ready to lay the terra cotta arches.

When a paneled ceiling is not undesirable, the arch does not need to be made the same depth as the supporting beams, provided it conforms to the other requirements, and details similar to those in Fig. 215 (*a*) and (*b*) may be used. The arch may be supported on shelf angles riveted to the beam, as in (*a*), or by terra cotta blocking and special skewbacks as in (*b*). In this way, the dead weight of the cinder fill over the arches may be reduced frequently by raising the top of the arch blocks level with the tops of the beams. A skewback, such as shown in (*e*), should not be used. Instead, it should have a web, as indicated by the dotted lines, to absorb the thrust.‡

Figure 215 (*f*) indicates the usual procedure when the beams are of varying depths. The typical beam is the controlling one, establishing the ceiling line, and the deeper beams are allowed to project into the room and form a paneled ceiling, or may be concealed in a partition below. Beams shallower than the typical, if used, should be placed with their bottom flanges flush with the typical beam and the space over them made up in fill. If it is desired to keep the depth of a beam, subjected to heavy loading, the same as that of the typical beams, two channels or two channels and a plate may be used, as shown at *A*. Special instances of construction are shown in Fig. 216.

Prob. 135b. Design a typical interior panel 18′-0″ × 18′-0″ of flat terra cotta arch construction. L.L. = 60#/□′. Use 3 sub-panels. Wood finish flooring.

Prob. 135c. Could a smaller floor beam be used theoretically in the interior panel of Prob. 135*a* than in an exterior panel? Determine the size of wall girder required.

136. Segmental Terra Cotta Arches.

The general description and the relative merits and uses of segmental arches as compared with flat arches are given in Art. 133. Figure 217 shows a typical section of such construction. The tile may be set in end bearing but this is not satisfactory unless the arches are of uniform spans and rises in all cases. For this reason, side construction is

* Some engineers use the distance from the center line of the rod to the top of the tile, and others use the distance corresponding to the top of the beam. See article "Tie Rods for Floor Arches," by F. N. Kneass, Engineering News Record, March 18, 1915, p. 518.

† Calculations based upon a uniform arrangement of tie rods across the whole floor.

‡ Arches have been known to fail on account of this web being omitted.

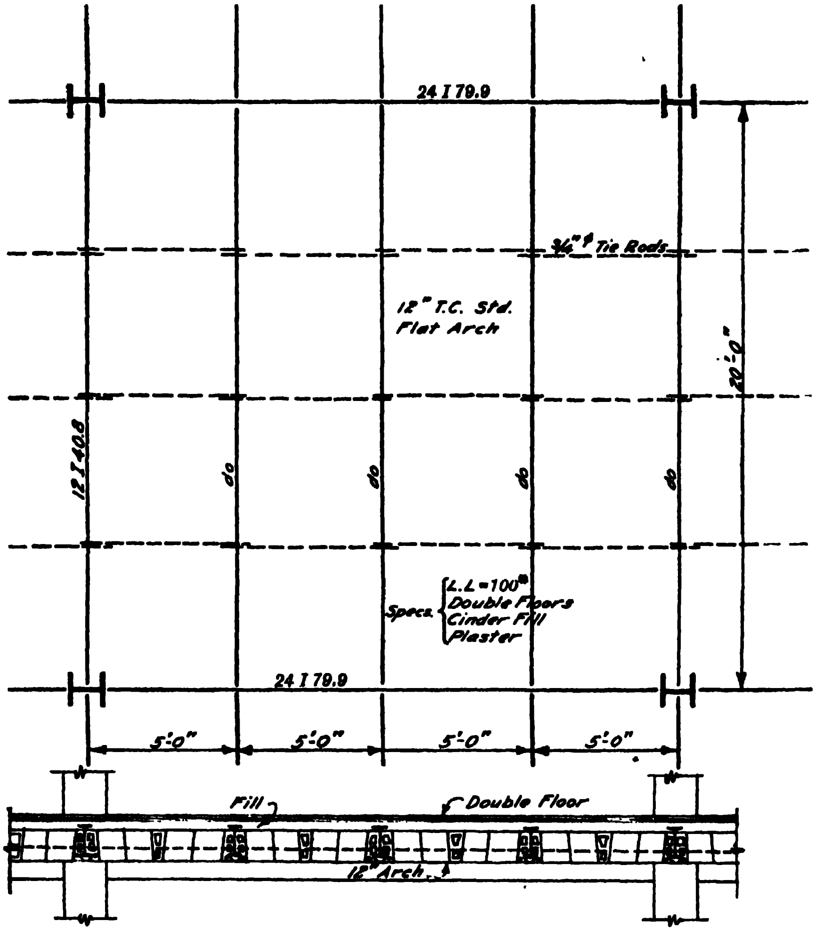


FIG. 212. TYPICAL TERRA COTTA ARCH FLOOR PANEL-STEEL FRAME

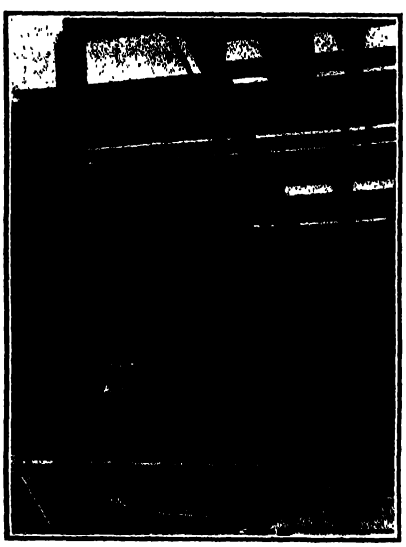


FIG. 213

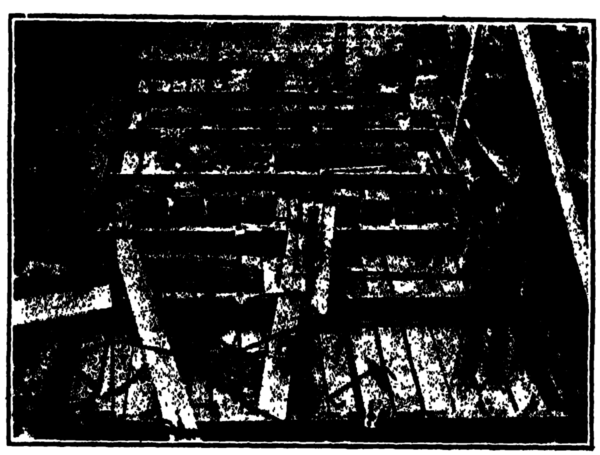


FIG. 214

preferable in the usual floor. If a flat ceiling is desired with this type of arch, a typical suspended ceiling may be used, but the merits of the flat arch should be investigated and compared with such a

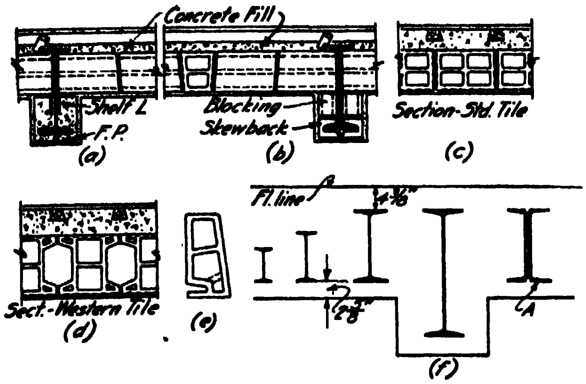


FIG. 215

case. Figure 217 shows only a medium span of such construction, but a segmental arch may be used for spans up to 25′-0″, although 16′-0″ represents the average. For spans less than 10′-0″, such construction is not economical. For econom-

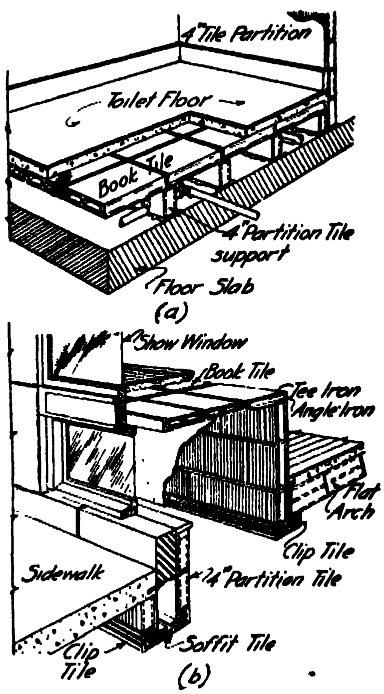


FIG. 216

ical segmental arch construction, no intermediate floor beams are used, and the supporting beams are placed on the column center-lines in a transverse direction, with tie beams on the column center-lines in the longitudinal direction of the building.

The depth of the arch should be sufficient to resist the thrust safely (Illustrative Prob. 134*a*). Table 55 gives the safe loads in #/□′ for various combinations of spans and arch depths. Sectional areas of segmental arches are given in Table 53. For reasons of stiffness and fire protection, a minimum arch depth is usually stipulated.

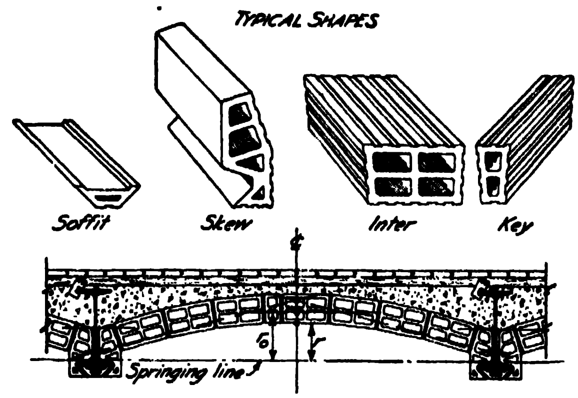


FIG. 217

SPECIFICATION CLAUSE*

Segmental arches shall have sufficient depth between the top and bottom faces to carry the load to be imposed, but not less than 6 inches. The tile shall have at least two cellular spaces in the depth.

For this reason, the values given in Table 55 only control for heavy loads. The 6″ arch is nearly as strong as the 8″, so that the former is used in the majority of cases.

To obtain the weight of the dead load per square foot, an average thickness of fill must be assumed since it varies from a minimum at the key to a maximum at the skewbacks. The weights of segmental terra cotta blocks are 20, 26, 32 and 40#/□′ for 4″, 6″, 8″ and 10″ respectively. The finish flooring may be estimated according to whether it is the typical wood floor with screeds, granolithic, tile, or marble. If a suspended ceiling is used, an allowance of 15#/□′ should be made for it.

The rise is commonly defined as the vertical distance between the springing line and the highest point of the concave surface (r in Fig. 217). It is the opinion of the authors that this value may be increased safely by one-third the depth of the arch, similar to the assumption made in other design work, and as illustrated by r_0 in Fig. 217. Other things being equal, the greater the rise of an arch, the less the thrust exerted and consequently the less the lateral resistance required of the supporting beams. However, the relations of the bottom of the skewbacks to the lower flanges of the beams, and

* The National Board of Fire Underwriters.

TABLE 55

SAFE LOADS IN #/□' FOR SEGMENTAL T.C. ARCHES*

Spans, Feet and Inches	Rise, In. per Ft. of Span	4-inch Arch	6-inch Arch	8-inch Arch	10-inch Arch	Spans, Feet and Inches	Rise, In. per Ft. of Span	4-inch Arch	6-inch Arch	8-inch Arch	10-inch Arch
11	¾	244	315	376	411	17	¾	151	194	232	254
	1	327	421	503	550		1	205	265	316	345
	1¼	404	519	621	678		1¼	256	330	394	430
	1½	479	617	737	805		1½	304	392	468	512
	1¾	551	709	847	925		1¾	351	452	540	590
	2	617	794	948	1036		2	393	506	605	661
11–6	¾	233	299	358	391	18	¾	141	182	218	238
	1	312	401	480	524		1	192	248	296	324
	1¼	388	499	596	652		1¼	240	310	370	404
	1½	460	592	707	773		1½	287	370	442	482
	1¾	528	680	812	887		1¾	330	425	507	554
	2	591	761	909	993		2	371	477	570	623
12	¾	222	285	341	372	19	¾	134	173	206	225
	1	297	383	458	500		1	181	233	279	304
	1¼	370	477	569	622		1¼	227	293	350	382
	1½	439	566	676	738		1½	271	348	416	455
	1¾	505	649	776	848		1¾	312	402	480	524
	2	565	727	869	949		2	351	451	539	589
12–6	¾	212	273	326	356	20	¾	126	163	194	212
	1	284	366	437	478		1	172	221	265	289
	1¼	354	456	545	595		1¼	215	277	331	361
	1½	420	541	646	706		1½	257	330	395	431
	1¾	483	621	742	811		1¾	296	381	455	497
	2	541	696	832	909		2	332	427	510	558
13	¾	203	261	312	341	21	¾	119	153	183	200
	1	272	351	419	458		1	163	209	250	273
	1¼	339	437	522	570		1¼	205	263	315	344
	1½	403	519	620	677		1½	243	314	375	409
	1¾	463	596	712	778		1¾	281	361	432	472
	2	521	670	801	875		2	315	406	485	530
14	¾	186	240	287	313	22	¾	113	145	174	190
	1	253	326	390	426		1	154	199	237	259
	1¼	315	406	485	530		1¼	194	250	298	326
	1½	374	482	575	629		1½	232	299	357	399
	1¾	430	553	661	722		1¾	268	344	412	450
	2	481	619	740	808		2	301	377	462	505
15	¾	174	225	268	293	23	¾	108	139	166	181
	1	234	302	361	394		1	147	190	227	247
	1¼	292	377	450	491		1¼	185	238	284	310
	1½	347	447	534	583		1½	221	284	340	371
	1¾	401	515	616	673		1¾	255	328	392	428
	2	449	577	690	754		2	286	369	440	481
16	¾	162	209	249	272	24	¾	102	132	157	172
	1	218	281	336	367		1	140	181	216	236
	1¼	274	353	421	460		1¼	177	227	272	297
	1½	325	419	500	546		1½	211	272	325	355
	1¾	374	481	575	628		1¾	244	314	375	410
	2	420	540	645	705		2	274	353	421	460

* The weight of the arch tile has been deducted so that only the D.L. of fill, plastering, and so on, must be deducted to obtain the net L.L. Factor of safety = 7.0.

the top of the arch to the tops of the beams, with the weight of the corresponding fill as a consideration, are important. A minimum fill of 2" over the tops of the beams is essential (Fig. 217). The rise should not be made too small as no great advantage would be gained by the use of segmental construction. Based upon such experience, the rise (actual) should be from $\frac{1}{10}$ to $\frac{1}{8}$ of the span, and $1\frac{1}{4}$" per foot of span is commonly specified.*

With such data established, the thrust may be calculated from

$$p = \frac{3\,w_0 \cdot L_a^2}{2\,r}.$$

The spacing of the tie rods should then be fixed, as described in Art. 134. The maximum value should not exceed one-half of the span of the arch. When no suspended ceiling is used, the tie rods must be covered with fire-resisting materials.

SPECIFICATION CLAUSE *

Steel tie-rods of proper size, spacing, and location shall be used in all arches to properly resist the thrust. Such tie rods shall be completely encased to a depth of at least 2 inches in fireproofing material which shall extend into and be anchored to the arch.

The size is usually $\frac{3}{4}$", and to be most effective, they should be located at the center of the skew blocks. The rods should be painted, the same as for all other structural steel. Figure 218 shows a common method of encasing tie rods or tie beams. The bending moments in the two respective directions may now be calculated and the beams proportioned for the maximum stresses, as described in Art. 134.

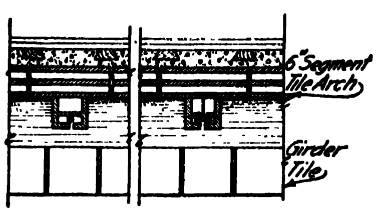


Fig. 218

Illustrative Prob. 136a. Design a typical panel 20'-0" × 20'-0" of segmental terra cotta arch construction for a L.L. of 200#/□'. Use one intermediate floor beam. No ceiling.

Span of arch = 10'-0". Assume 6" Arch.

Rise $1\frac{1}{4}$" per foot = $1\frac{1}{4} \times 12$ = 15" actual.

L.L. =	200
Fin. Flr. =	3
Sub Flr. =	3
Fill =	50 (average)
6" Arch =	26
T.L. =	282#/□'

$$p = \frac{3\,w_0 \cdot L_a^2}{2\,r} \quad w_0 \text{ based on outside panel thrust.}$$

Theoretical rise = $20 + \frac{6}{3}$ = 22.0"

$$p = \frac{3 \times 282 \times (10)^2}{2 \times 22} = 1920\#/\text{ft.}$$

$$L_s = \frac{4800}{1920} = 2.5'$$

Try $\frac{7}{8}$" ϕ tie rods.

$$T_r = 0.419\square'' \times 16{,}000 = 6700$$

$$L_s = \frac{6700}{1920} = 3.5'$$

Use $\frac{7}{8}$" ϕ tie rods, 4'-0" o.c.

Sectional area 6" arch = 36□"

$\frac{1920}{36}$ = 58#/□" compression O.K.

Load per foot on beam = 282 × 10 = 2820#/ft.
Beam and soffit block = 100
T.L. = 2920

$$M_{1\text{-}1} = 1.5 \times 2920 \times (20)^2 = 1{,}752{,}000''\#.$$

$$\frac{I}{c} = \frac{1{,}752{,}000}{16{,}000} = 109.3''^3 \text{ (Trial)} \qquad \text{Try 20 I 65.4}$$

$$M_{2\text{-}2} = 1.0\,p \cdot L_s^2 = 1.0 \times 1920 \times (4)^2 = 30{,}600''\#$$

$$s_{1\text{-}1} = \frac{1{,}752{,}000}{117.0} = 14{,}970\#/\square''$$

$$s_{2\text{-}2} = \frac{30{,}600}{8.9} = 3{,}440$$

18,410#/□" *Use 20 I 65.4*

Use 6" T.C. Segmental Arches
Rise = 1'-3"

Concentration on girder = 2820 × 20 = 56,400#

Moment due to concentration at mid-span

$$M = \frac{P \cdot L}{4} = \frac{56{,}400 \times 20}{4} \times 12 = 3{,}383{,}000''\#$$

Girder weight and F.P. assumed as 150#/ft.

$$M = 1.5\,w \cdot L^2 = 1.5 \times 150 \times (20)^2 = 90{,}000$$

Total = 3,473,000"#

$$\frac{I}{c} = \frac{3{,}473{,}000}{16{,}000} = 216.4''^3$$ *Use 24 I 105 Girders.*

Prob. 136b. Design a typical panel of segmental terra cotta arch construction, 16'-0" × 18'-0" for a L.L. of 150#/□'. Use no intermediate floor beams. No ceiling.

137. Reinforced Terra Cotta Arches.†

There are several systems of floor construction which may be properly included with terra cotta arches, in which the spaces between the tile are usually reinforced with metal in one form or another. The shapes of the blocks vary according to the particular system. Many of these methods of construction are patented.

One of these systems is the New York Reinforced Flat Arch, as illustrated in Fig. 219 (*a*). It is simply a form of end construction flat arch, bearing

* National Board of Fire Underwriters.

† For a more detailed description of such arches, see Kidder's Architects' and Builders' Pocket Book, Freitag's "Fire Prevention and Fire Protection," — John Wiley & Sons, Inc., and Bulletins of National Fire-Proofing Co., as well as literature for the patented types.

the Bevier patent, in which a wire truss reinforcement is used. The insert shows the variations for either a suspended or panelled ceiling. A newer adaptation of this construction is shown in Fig. 219 (*b*), which consists of self-centering units which are set dry, except the skewbacks, permitting installation by unskilled labor. The lipped tile forms channels for the cement mortar ribs 1" wide in which $\frac{1}{4}$" or $\frac{3}{8}$"ϕ rods are imbedded. An advantage is that a mortar key is obtained instead of using a key block.

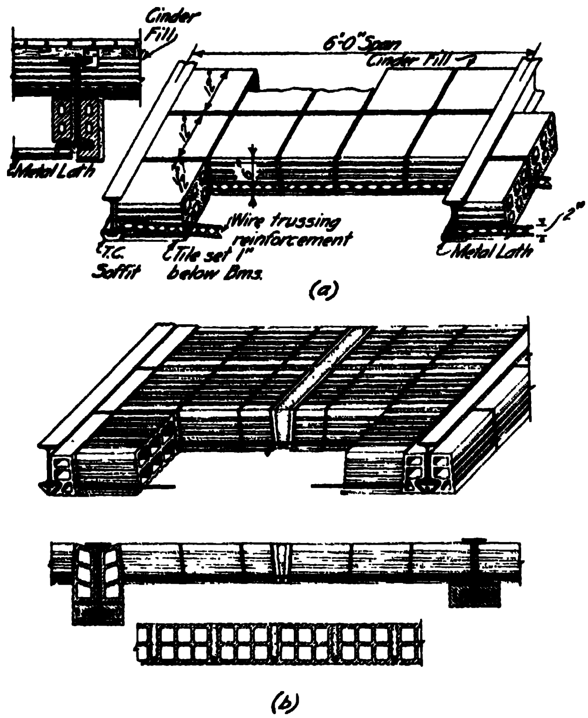


FIG. 219

Another patented method is called the Johnson System Floor, in which a wire fabric is placed under the tile and over the entire floor area with $\frac{1}{2}$" ϕ rods, 4" o.c., running lengthwise. All this steel is imbedded in a layer of mortar, as shown in Fig. 220. The tile may be set as shown, or run over the tops of the I-beams, or set flush with the bottoms of the beams and a fill used. A special herringbone ceiling of tile may be used if desired. A **combination construction** is sometimes employed by using segmental terra cotta arches (Art. 136) in the interior panels and flat arches to take up their thrusts in the exterior panels (usually of the " New York " type).

Another form of construction used for floors, particularly when vaulted ceilings are desired, is the Guastavino arch, as shown in Fig. 221. This is made up of thin Rumford clay tiles 1" thick, 6" wide, and from 12" to 24" long, in two, three, or more layers according to the requirements, and all bonded together. Figure 222 shows an actual adaptation of this construction for a floor.

Other types of patented arches are the Herculean and the Excelsior, as shown in Fig. 223 (*a*) and (*b*), respectively. In the former, the terra cotta blocks have grooves in which $1\frac{1}{2}" \times 1\frac{1}{2}" \times \frac{3}{16}"$ T's are grouted into place. In the latter, special shaped tiles are used which provide a clear space for the insertion of the tie rods without cutting the blocks.

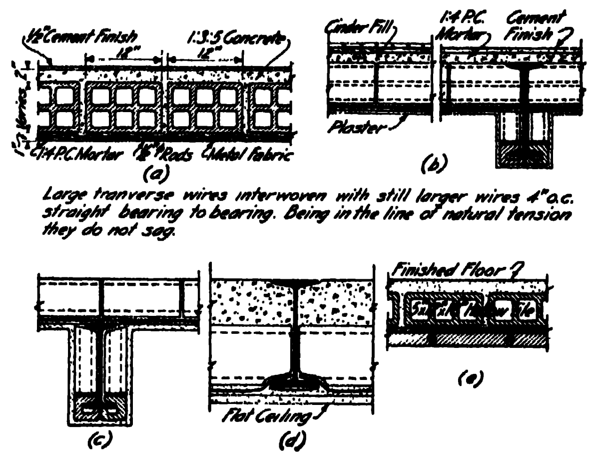


FIG. 220

138. Brick Floor Arches.

The brick floor arch, which was formerly used quite extensively, is **seldom employed** in modern practice, except for the possibility of using it in mill buildings or warehouses where extremely heavy loads are to be carried. The decided advantages which terra cotta floor arches offer have caused the use of brick to become nearly obsolete. Figure 224 shows a section through a typical floor, although terra cotta skewbacks may be used to fully protect the steel beams. This type of arch is one of the strongest of the varying kinds that can be built and it will carry load well even when subjected to considerable deflection, but it is heavier and more expensive to build than other types of construction.

SPECIFICATION CLAUSE*

> Segmental arches of brick shall have a thickness of not less than 4 inches for spans of 5 feet or less, and 8 inches for spans exceeding 5 feet and not exceeding 8 feet. Brick arches shall be composed of good, hard common or hollow brick. The brick shall be laid to a line on the centers and properly and solidly bonded; each longitudinal line of brick shall break joints with the adjoining lines. The arches shall spring from suitably designed solid skewbacks made of the same materials as the arches, and be properly keyed. The brick shall be well wet before laying, and the joints solidly filled with mortar.

The rise should not be less than $1\frac{1}{4}$" per foot of span. Tie rods should be used as in other arches. The design of such construction is similar to that of small segmental terra cotta arches, as discussed in Art. 134.

* National Board of Fire Underwriters.

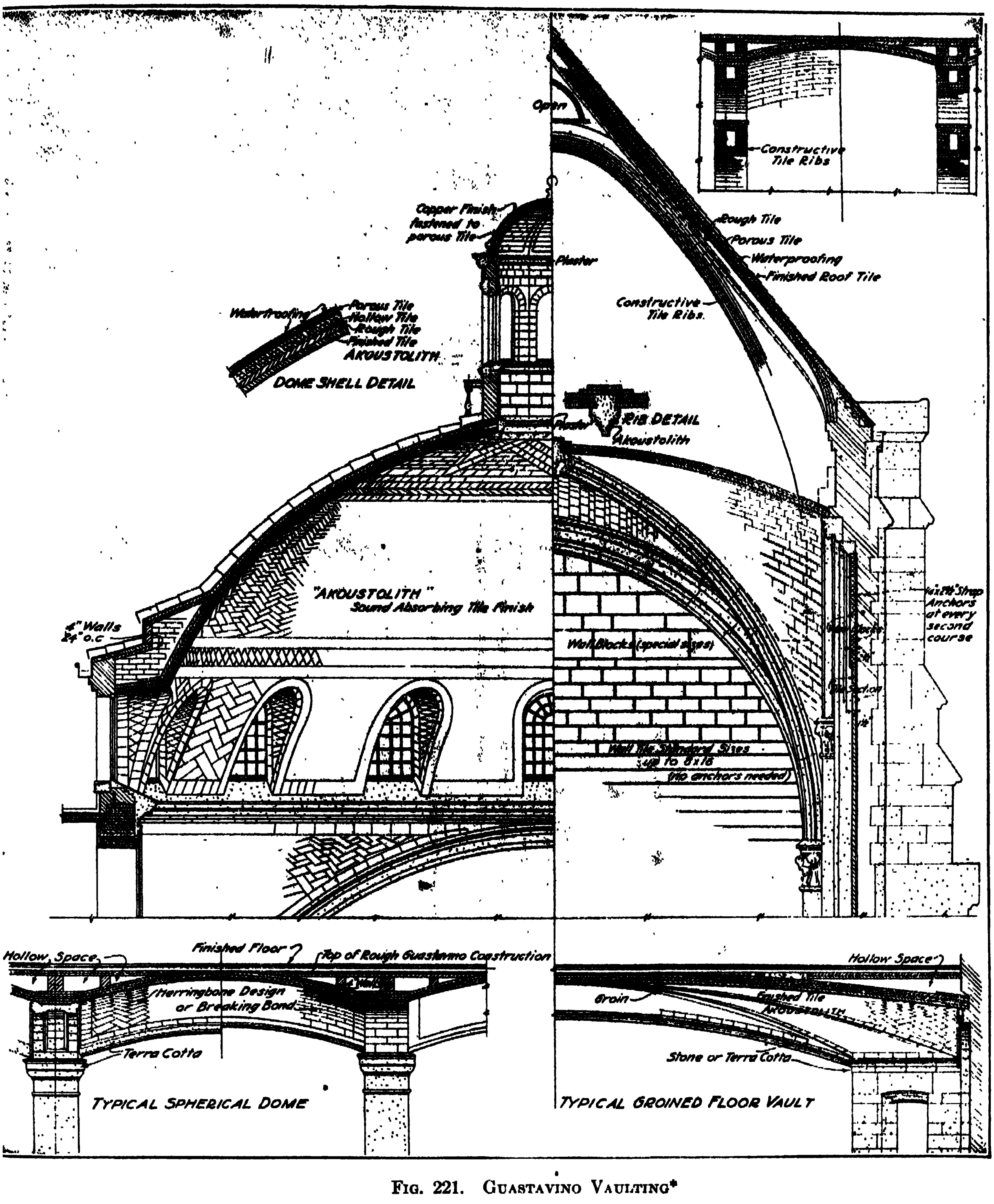
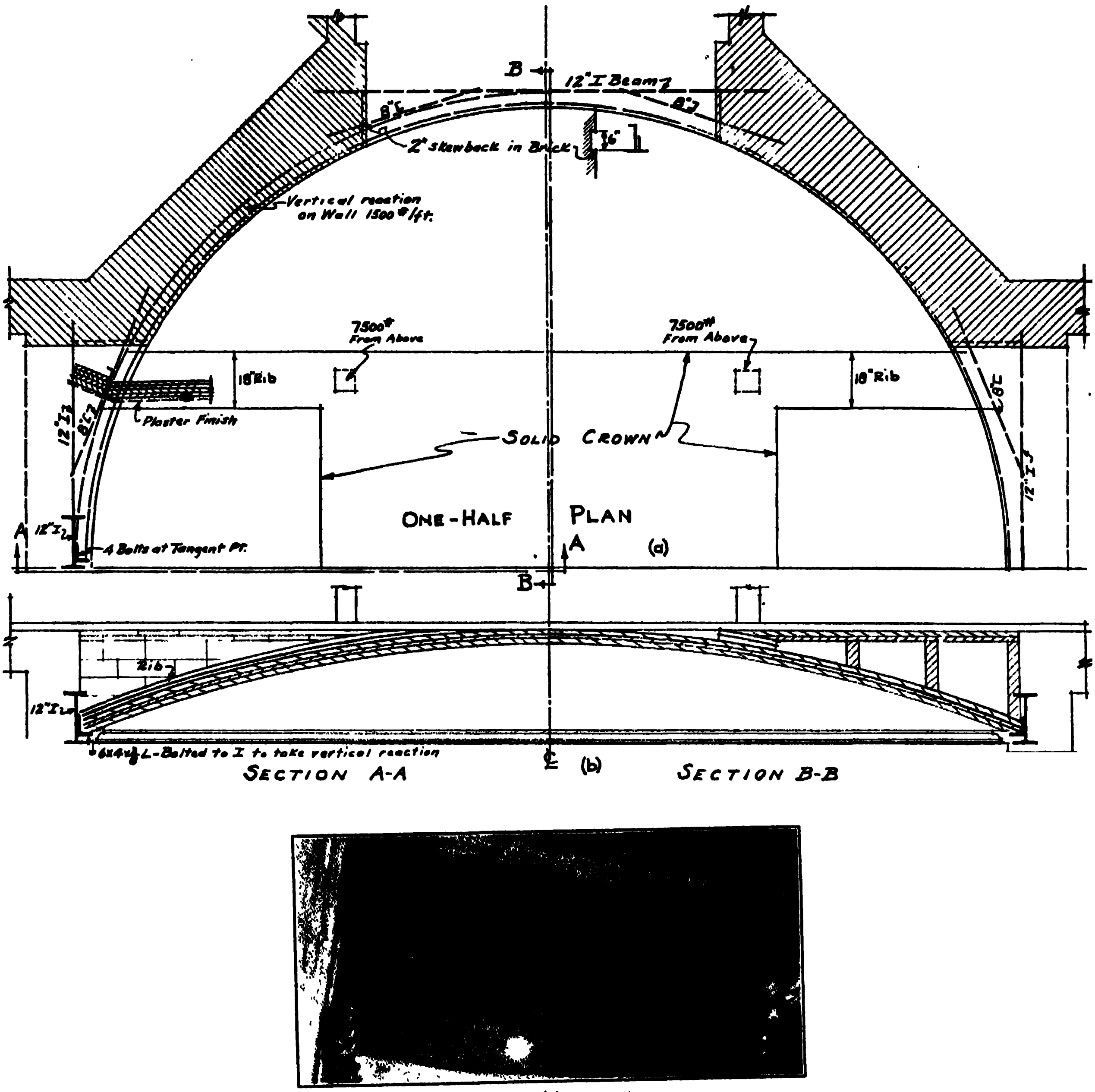


FIG. 221. GUASTAVINO VAULTING*

* Courtesy R. Guastavino Co.



(c)

Fig. 222*

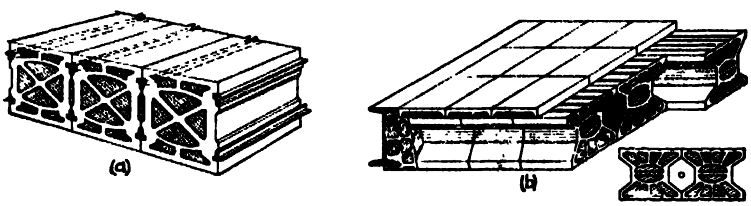


Fig. 223

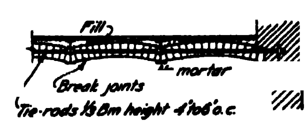


Fig. 224

* Courtesy of R. Guastavino Co.

Section 9b

CONCRETE SEGMENTAL ARCHES

139. Typical Construction.

Another form of segmental arch construction which embodies a concrete carrying floor with a steel frame is the segmental arch. It is particularly adaptable to floors carrying heavy loads such as for warehouses, or to floors subject to the vibration of machinery. In both of these instances, a flat ceiling is not usually required, although such a ceiling may be used by suspending it in the usual way. An arched slab is preferable to a flat slab because the concrete is most effective in this form and therefore less depth is required. Figure 225 illustrates the typical construction. Permanent expanded metal centering is generally used, as it may be bent to the curve of the arch, and the cost of forms is thereby materially reduced.

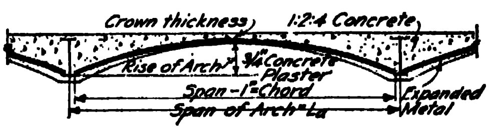


Fig. 225

The thrust may be calculated in the manner discussed in Art. 134 by the use of the formula

$$p = \frac{3\,w_0 \cdot L_a^2}{2\,r} \quad (\#/\text{linear ft.}).$$

The rise of the arches (r inches) is often specified as 1" per foot of span, but it should preferably not be less than one-eighth of the span. The spans, L_a, usually vary from 6'-0" to 12'-0". As the haunches are not as rigid as they might be, it is conservative to use a liberal depth, d, at the crown, and a minimum of 4" is usually specified for the overall thickness here. The required depth may be calculated from the usual procedure of dividing the force by the allowable stress to obtain the required area, or

$$\text{Crown depth} = \frac{\text{thrust per linear foot}}{12 \times \text{the allowable comp. stress}},$$

or

$$d = \frac{p}{12 f_c}.$$

Stone concrete is often used for such construction, because of its greater compressive strength, although cinder concrete may be used for the lighter loads. Tie rods should be used in a manner similar to that described for segmental terra cotta arches (Art. 136). The design of the steel beams is also based upon the principles discussed in Art. 134 for tile arches. The size of the floor beam selected must be such that the arch has its required rise and so that a minimum of 2" of fire protection is provided over the tops of the beams. Table 56 may be used to eliminate some of the computations in many cases. Table 57 is also valuable in determining the weight per square foot of floor for the arches, averaged over the entire span.

Illustrative Prob. 139a. Design a typical panel, 20'-0" × 20'-0", for a L.L. of 300#/□', using segmental concrete arch construction. 1" granolithic finish floor. Use 10'-0" arch spans.

L.L. = 300 Rise = $\frac{1}{8} \times 10 \times 12 = 15''$
1" Grano. = 12
Arch = 107 (Table 57)
T.L. = $\overline{429}$#/□' say 430

$$p = \frac{3\,w_0 \cdot L_a^2}{2\,r} = \frac{3 \times 430 \times (10)^2}{2 \times 15} = 4300\#/\text{ft.}$$

$$d = \frac{4300}{12 \times 300} = 1.3''$$

Use 4" crown thickness. (Table 56.)
Use #24 Self-Sentering.

Load per foot on beam = 429 × 10 = 4290#/ft.
Bm. and F.P. = 110
T.L. = $\overline{4400}$#/ft.

$$M_{1\text{-}1} = 1.5\,w \cdot L^2 = 1.5 \times 4400 \times (20)^2 = 2{,}640{,}000''\#$$

$$\frac{I}{c} = \frac{2{,}640{,}000}{16{,}000} = 165''^3 \qquad \text{Try 20 I 100.}$$

$$L_s = \frac{A_s \cdot f_s}{4300} = 4 \times 4300 = A_s \times 16{,}000$$

A_s = 1.08□" Use $2\frac{1}{2} \times 2 \times \frac{3}{8}$ ∠ ties with bolted connections 4'-0" o.c.

$$M_{2\text{-}2} = 1.0 \times 4300 \times (4)^2 = 68{,}800''\#$$

$$s_{1\text{-}1} = \frac{2{,}640{,}000}{165.6} = 15{,}940$$

$$s_{2\text{-}2} = \frac{68{,}800}{14.5} = 4{,}730$$

$\overline{20{,}670}$#/□" Too high

Try 20 BG 112

$$s_{1\text{-}1} = \frac{2{,}640{,}000}{234.2} = 11{,}230$$

$$s_{2\text{-}2} = \frac{68{,}800}{39.8} = 1{,}730$$

$\overline{12{,}960}$#/□"

Use 20 BG 112.

Concentration on girder = 4400 × 20 = 88,000#

$$M = 3\,P \cdot L''\# = 3 \times 88{,}000 \times 20 = 5{,}280{,}000''\#$$

$$\frac{I}{c} = \frac{5{,}280{,}000}{16{,}000} = 330''^3$$

Use 26 BG 150 *or* 30 BI 120.

From the above illustration, it should be evident that such a form of floor construction is not always economical of materials, especially when compared with solid concrete or ribbed slabs.

TABLE 56*

CROWN THICKNESSES FOR CONCRETE SEGMENTAL ARCHES

Rise in In.	Live Loads in Pounds per Sq. Ft. 300 Stone	300 Cinder	200 Stone	200 Cinder	150 Stone	150 Cinder	100 Stone	100 Cinder	Length of Sheet
11'-0" Span									
12	5"	6½"	4"	5"	3½"	4"	3"	3¼"	11'-2"
15	4"	4¾"	3½"	3¾"	3"	3½"	3"	3"	11'-4"
18	3½"	3¾"	3"	3¼"	3"	3"	3"	3"	11'-6"
21	3"	3½"	3"	3"	3"	3"	3"	3"	11'-8½"
24	3"	3½"	3"	3"	3"	3"	3"	3"	11'-11"
10'-0" Span									
12	4½"	5½"	4"	4¼"	3½"	3¾"	3"	3¼"	10'-2½"
15	3½"	4"	3"	3½"	3"	3"	3"	3"	10'-4½"
18	3"	3½"	3"	3"	3"	3"	3"	3"	10'-6¾"
21	3"	3"	3"	3"	3"	3"	3"	3"	10'-9½"
24	3"	3"	3"	3"	3"	3"	3"	3"	11'-0"
9'-0" Span									
12	3"	5"	3"	4¼"	3"	3½"	3"	3"	9'-3"
15	3"	4"	3"	3½"	3"	3"	3"	3"	9'-5"
18	3"	3¼"	3"	3"	3"	3"	3"	3"	9'-7½"
21	3"	3"	3"	3"	3"	3"	3"	3"	9'-10½'
24	3"	3"	3"	3"	3"	3"	3"	3"	10'-1½"
8'-0" Span									
9	3"	5"	3"	4"	3"	3½"	3"	3"	8'-1½"
12	3"	4¼"	3"	4"	3"	3"	3"	3"	8'-3½"
15	3"	3¾"	3"	3½"	3"	3"	3"	3"	8'-5¾"
18	3"	3"	3"	3"	3"	3"	3"	3"	8'-8½"
21	3"	3"	3"	3"	3"	3"	3"	3"	8'-11¾"
24	3"	3"	3"	3"	3"	3"	3"	3"	9'-3½"
7'-0" Span									
9	3"	5"	3"	3¾"	3"	3¼"	3"	3"	7'-2"
12	3"	4"	3"	3½"	3"	3"	3"	3"	7'-4"
15	3"	3½"	3"	3"	3"	3"	3"	3"	7'-6¾"
18	3"	3"	3"	3"	3"	3"	3"	3"	7'-9¾"
21	3"	3"	3"	3"	3"	3"	3"	3"	8'-1½"
24	3"	3"	3"	3"	3"	3"	3"	3"	8'-5½"
6'-0" Span									
9	3"	4½"	3"	3½"	3"	3"	3"	3"	6'-2"
12	3"	3½"	3"	3½"	3"	3"	3"	3"	6'-4½"
15	3"	3"	3"	3"	3"	3"	3"	3"	6'-7½"
18	3"	3"	3"	3"	3"	3"	3"	3"	6'-11"
21	3"	3"	3"	3"	3"	3"	3"	3"	7'-3"
24	3"	3"	3"	3"	3"	3"	3"	3"	7'-7½"

* Based upon the General Fireproofing Co. handbook.

TABLE 57†

WEIGHT IN #/□' OF STONE CONCRETE SEGMENTAL ARCHES (AVER. FOR SPAN)‡

Rise	Thickness of Slab at Crown of Arch 7½"	7"	6½"	6"	5½"	5"	4½"	4"	3½"	3"
11'-0" Span										
12"	140	134	128	122	116	110	104	98	92	86
15"	149	143	137	131	125	119	113	107	101	95
18"	161	155	149	143	137	131	125	119	113	107
21"	172	166	160	154	148	142	136	130	124	118
24"	183	177	171	162	159	153	147	141	135	129
10'-0" Span										
12"	140	134	128	122	116	110	104	98	92	86
15"	149	143	137	131	125	119	113	107	101	95
18"	161	155	149	143	137	131	125	119	113	107
21"	171	165	159	153	147	141	135	129	123	117
24"	180	174	168	162	156	150	144	138	132	126
9'-0" Span										
12"	138	132	126	120	114	108	102	96	90	84
15"	149	143	137	131	125	119	113	107	101	95
18"	160	154	148	142	136	130	124	118	112	106
21"	171	165	159	153	147	141	135	129	123	117
24"	180	174	168	162	156	150	144	138	132	126
8'-0" Span										
9"	127	121	115	109	103	97	91	85	79	73
12"	138	132	126	120	114	108	102	96	90	84
15"	149	143	137	131	125	119	113	107	101	95
18"	159	153	147	141	135	129	123	117	111	105
21"	170	164	158	152	146	140	134	128	122	116
24"	179	173	167	161	155	149	143	137	131	125
7'-0" Span										
9"	126	120	114	108	102	96	90	84	78	72
12"	138	132	126	120	114	108	102	96	90	84
15"	149	143	137	131	125	119	113	107	101	95
18"	159	153	147	141	135	129	123	117	111	105
21"	168	162	156	150	144	138	132	126	120	114
24"	177	171	165	159	153	147	141	135	129	123

† Based upon the General Fireproofing Co. handbook.
‡ For cinder concrete reduce by the ratio of 108 to 150.

There are several types of patented floor systems which are of the concrete segmental arch order. Among these are the Roebling, illustrated in Fig. 226. Other combinations may be improvised by the use of tees or small channels and wire mesh without infringing upon the patent rights, but the patented types are probably cheaper as they are standardized.

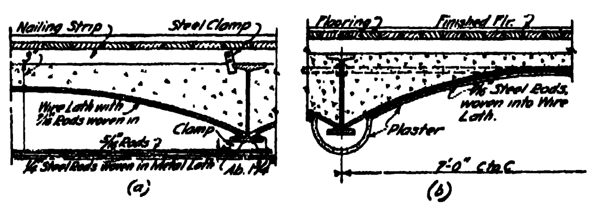


FIG. 226

Prob. 139b. Design a typical interior panel 16'-0" × 16'-0" for a L.L. of 200#/□', using segmental concrete arches with 8'-0" spans. Double wood floor. Use cinder concrete.

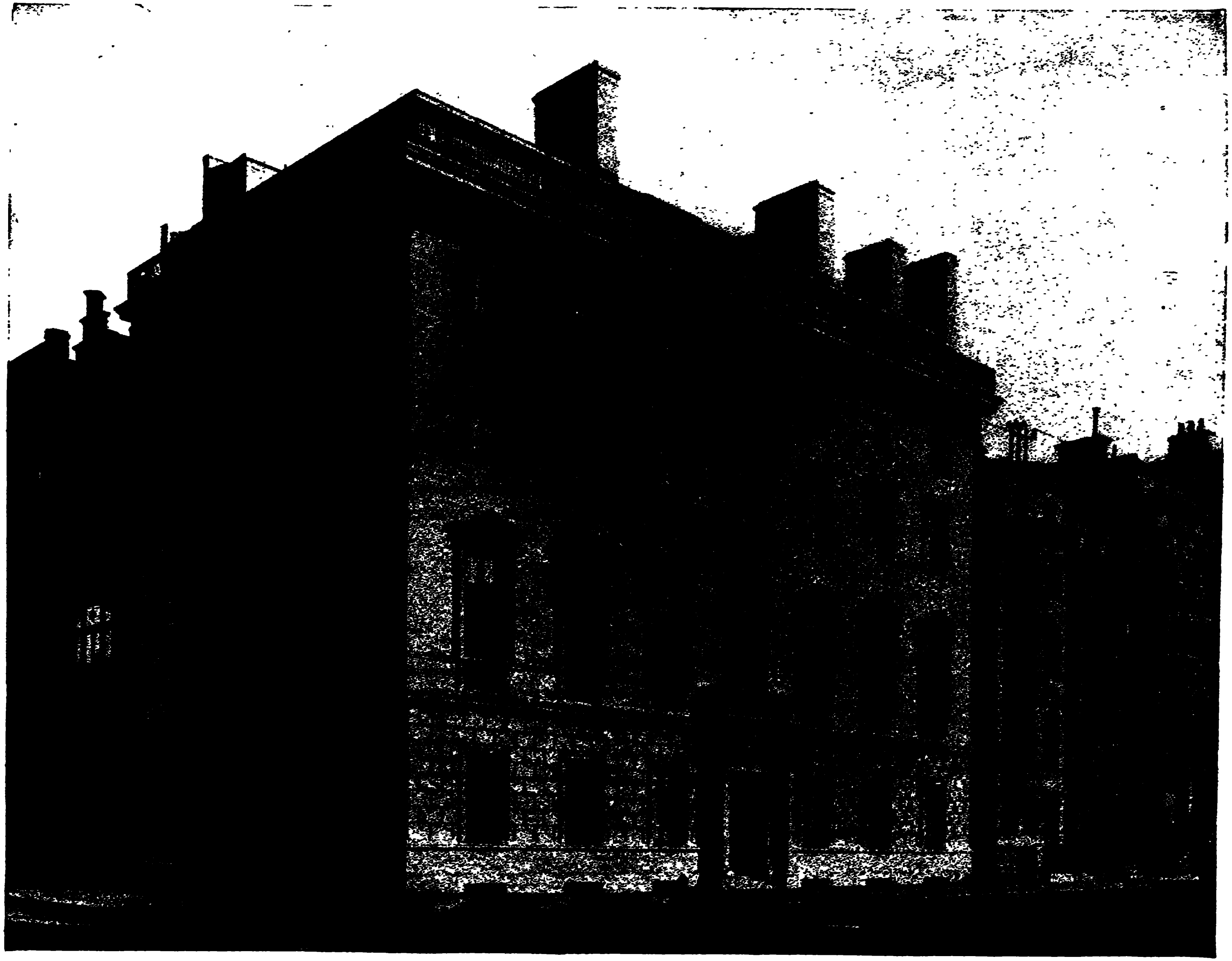


PLATE 17 THE CITY HOUSE
GENERAL VIEW OF EXTERIOR
GUY LOWELL, ARCHITECT

Photograph by M. E. Hewitt Studio

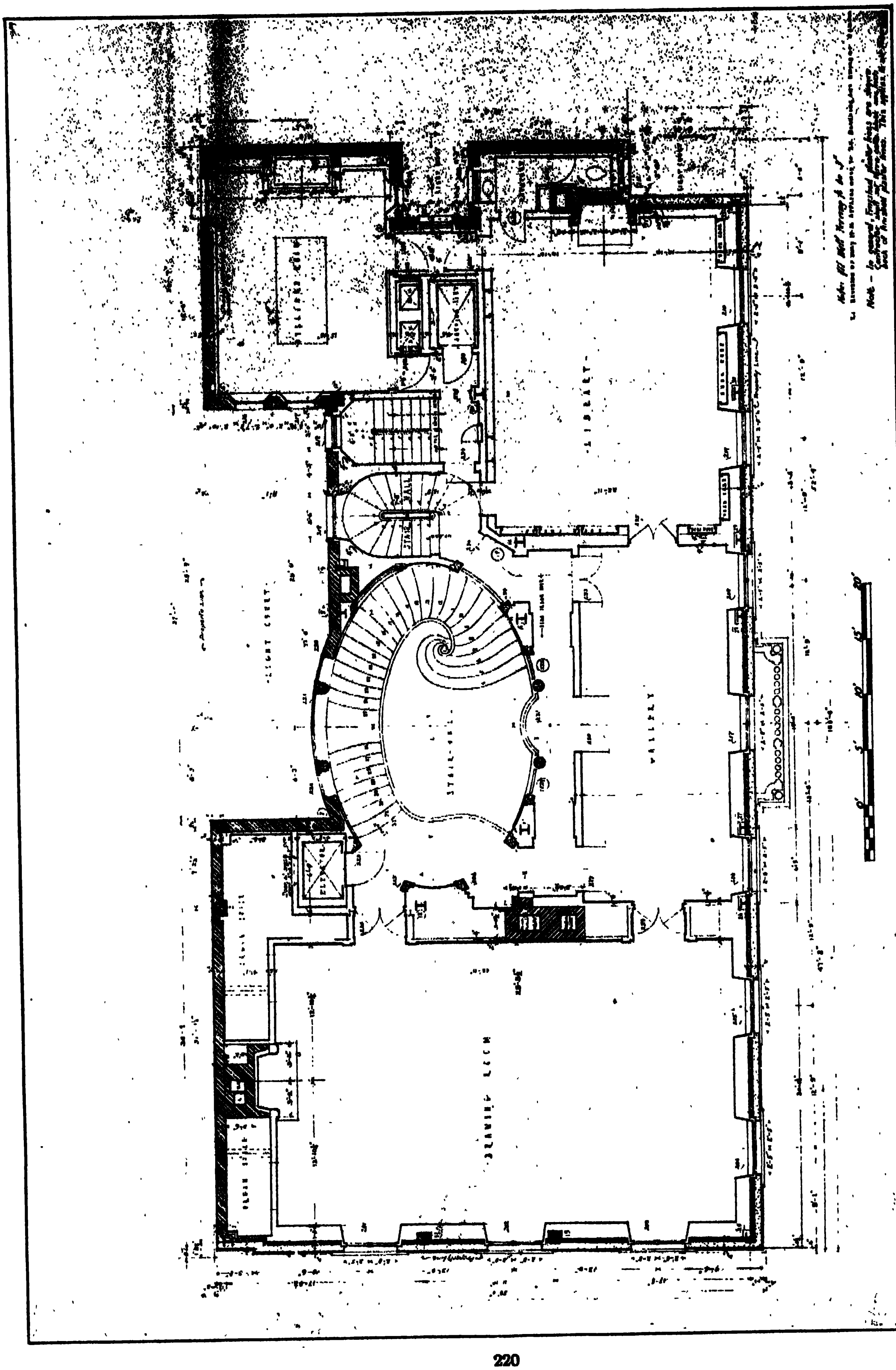


PLATE 18. THE CITY HOUSE
SECOND FLOOR PLAN
GUY LOWELL, ARCHITECT

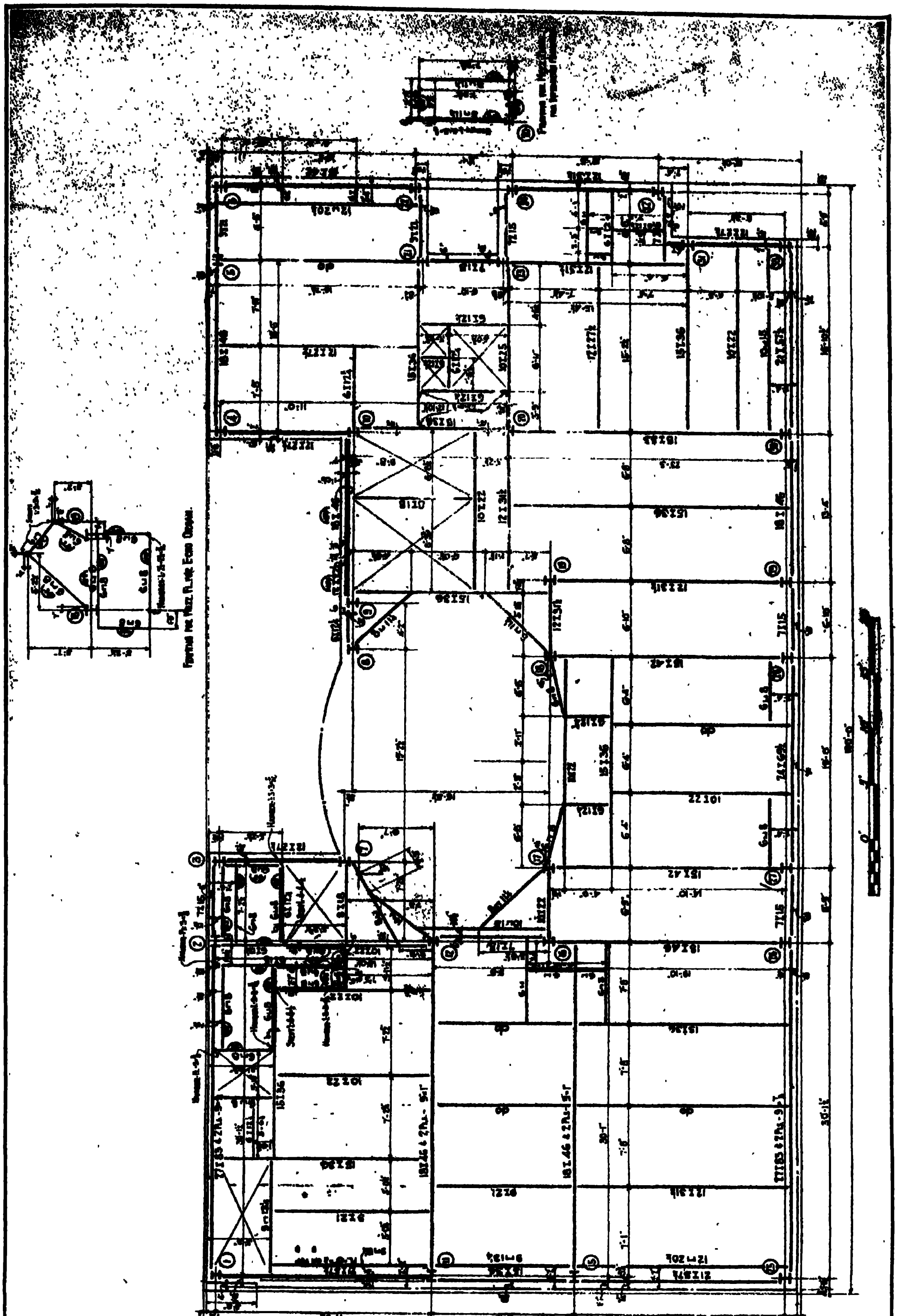


PLATE 19 THE CITY HOUSE
SECOND FLOOR FRAMING PLAN
GUY LOWELL, ARCHITECT

CHAPTER 10

SOLID SLAB CONSTRUCTION

140. General Considerations.

One form of fire-resisting slab construction is that in which concrete slabs constitute the carrying floor and steel beams and girders the supporting frame.* The concrete may be made with cinders as an aggregate (Sect. 10A), or with stone as the coarse material (Sect. 10B), depending upon the advantages and local conditions surrounding the problem. Since the concrete slabs, if properly built, are considered to be fire-resisting, the steel beams and girders must be encased in fire-protecting materials to make the construction consistent as a whole, as structural steel, unprotected, is a poor material to successfully resist the action of a fire. (See Part VI.)† For such construction, concrete is the natural material and conforms with that of the slabs, although terra cotta blocks may be used in special instances (Fig. 206 (*b*)).

SPECIFICATION CLAUSES‡

Protection of Steel Beams and Girders The protection of the webs and bottom flanges of girders, and all members of trusses shall have a thickness of not less than 2 inches at all points. The protection of the webs and bottom flanges of beams, lintels, and all other structural members shall be not less than $1\frac{1}{2}$ inches at any point.

Concrete protection for all structural members shall be held in position by suitably designed interior steel anchors hooked securely around the flanges or angles of the members, at intervals not exceeding 8 inches apart; these anchors shall be not less than $\frac{1}{8}$ inch in thickness if flat or $\frac{1}{16}$ inch in diameter if of wire, and shall be located at a distance not less than $\frac{3}{4}$ inch, nor more than 1 inch from the outside surface. Provision shall be made to prevent displacement of anchors while concrete is being deposited. When the flange width of steel members exceeds 6 inches, the wire used for anchoring the concrete protection shall be not less than $\frac{1}{8}$ inch diameter.

Additional weight must be carried by the steel beams, and an allowance for such fire protection should be made in the design calculations. The rigid attachment of the material is accomplished by wrapping the bottom flange with wire mesh and by the use of vertical wires at the beam sides, as indicated in Fig. 227. The beam-side forms are sometimes slightly battered, to allow easier stripping of the forms. It is sometimes wisely specified that the bottom flanges of plate girders should be protected with 3″ of material on account of the large tributary floor areas which they support, and because of their deep projection into the room

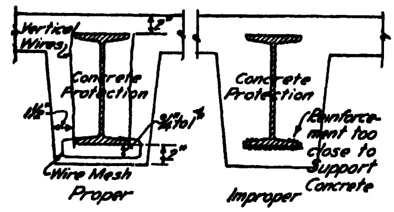


FIG. 227

Illustrative Prob. 140a. What allowance should be made for the weight of the fire-protecting materials (haunch) of a 15 I 42.8 (assumed)? 4″ Slab. Cinder concrete.

Flange width, $b = 5\frac{1}{2}''$.
Average width of protection (F.P.) $= 5\frac{1}{2}'' + 4'' = 9\frac{1}{2}''$ (allow 1″ bevel).
Depth of stem (part below slab) $= 15'' - 2'' + 2'' = 15''$.
Section area $= 15'' \times 9\frac{1}{2}'' = 142\square''$.
$\frac{2}{3}$ area of I-beam assumed to be taken out $= \frac{2}{3} \times 12.48$ (section area of beam) $= 8\square''$.
Net area $= 142 - 8 = 134\square''$.

$\frac{134}{144} \times 108$ 100#/ft. for haunch
42 for beam
142#/ft. beam and haunch.

Prob. 140b. What is the weight per linear foot for an 18 I 55 with a cinder concrete haunch and 4″ slab?

There are also a number of patented systems on the market, using pre-cast concrete sections placed between the steel beams (Art. 146). Another form of construction occasionally used is that of employing light steel members between the beams which serve as a form for the slab (Art. 147). Slabs of gypsum, either pre-cast or cast-in-place, are also a comparatively recent type of floor construction with a basic steel frame.

The typical plank floor, no matter how supported, may also be considered broadly as an elementary type of solid slab construction.

* Possible forms of construction, although not as common, are the use of concrete slabs, concrete floor beams and steel girders, or two-way concrete slabs supported by steel girders on four sides.

† Even in so-called non-fireproof buildings, it is good practice to fireproof beams supporting important parts of the structure, such as beams supporting bearing walls or columns. Even if concrete fireproofing is not used, metal lath and 1″ cement plaster is desirable.

‡ The National Board of Fire Underwriters.

141. Use of Wood Carrying Floors with Steel Frame.

A carrying floor of wood, such as a plank floor, combined with steel beams and girders, is naturally not considered first class construction. Plank or laminated floors, either laid flat or laminated, spanning between steel girders, with no intermediate supporting beams, are sometimes used in certain types of **mill construction.** Such framing should be used only with restrictions and built under the control of rigid specifications. It is almost always confined in its use to factory floor construction. Occasionally for **semi-mill construction,** plank floors, wood floor beams and steel girders are used. This type is also subject to rather confining limitations. Framing of both of these types is discussed in Book 1.*

Section 10A

CINDER CONCRETE SLABS

142. Typical Construction.

Cinder concrete slabs are quite commonly used with a steel frame because of the relatively light weight, both of the slabs and of the beam protection, and also on account of the minimum requirements of slab thicknesses and limiting spans. The latter are stipulated in order to assume proper limitations upon a material which has a lower compressive strength than stone concrete, and to provide minimum fire protection.

SPECIFICATION CLAUSE†

> Cinder concrete slabs shall not be less than four inches thick; they shall not exceed eight feet in span.

When the span exceeds 6′-0″, special care must be exercised in the inspection of the details of construction and the removal of forms. For this reason, designers avoid spans greater than this, and thereby often effect greater economy. Some specifications require that the thickness of the slabs shall not be less than $\frac{1}{18}$ of the span. Figure 228 illustrates the

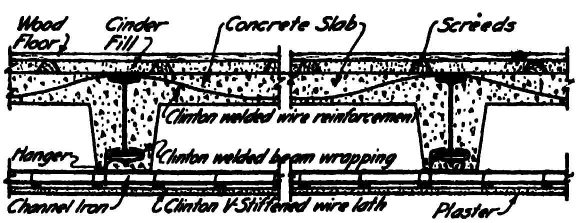


Fig. 228

typical construction. Wire reinforcement (Art. 106) is the more commonly used for such work, as only small amounts of reinforcement are required for medium spans, as the strength of the cinder concrete itself limits the strength of the combination. Light rods are relatively expensive on account of the increases over the base price (Art. 104) and this is another reason in favor of wire reinforcement.

The weight of the dead load per square foot of floor may be obtained in the usual way by reference to Table 30. It should be remembered that the values of f_c, f_s, n, p, k, j and K, in the design formulas vary considerably from those for stone concrete (Art. 103).

SPECIFICATION CLAUSE‡

> The allowable extreme fibre stress in compression in cinder concrete slabs between steel beams shall not exceed 300#/□″. The ratio of the moduli of elasticity of 1 : 2 : 5 cinder concrete and steel shall be taken as 1 to 30.

The mix of cinder concrete should never be leaner than 1 : 2 : 5. In common instances 1 : 2 : 4 is the mix used, in which case the value of f_c may be increased (Table 33). In order to obtain reliable concrete, the specifications usually require clean, well-burned, steam-boiler cinders. The tension reinforcement should never be less than 0.12%.

The value of f_s is usually taken as 18,000#/□″ (Art. 102), because of the wire used.

Illustrative Prob. 142a. Determine the value of K for f_c = 300#/□″ and f_s = 18,000#/□″.

$$n = 30$$

$$p = \frac{0.5}{\frac{f_s}{f_c}\left(\frac{f_s}{n \cdot f_c} + 1\right)} = \frac{0.5}{\frac{18{,}000}{300}\left(\frac{18{,}000}{30 \times 300} + 1\right)} = 0.0028$$

$$k = \sqrt{2\,p \cdot n + (p \cdot n)^2} - p \cdot n$$

$$\begin{aligned} p \cdot n = 0.0028 \times 30 &= 0.084 \\ 2\,p \cdot n &= 0.168 \\ (p \cdot n)^2 &= 0.007 \\ \text{Sum} &= 0.175 \\ \sqrt{\text{Sum}} &= 0.420 \\ p \cdot n &= 0.084 \\ k &= 0.336 \end{aligned}$$

$$j = 1 - \tfrac{1}{3}\,k = 1.000 - \frac{0.336}{3} = 0.888$$

$$K = \tfrac{1}{2} f_c \cdot k \cdot j = \tfrac{1}{2} \times 300 \times 0.336 \times 0.888 = 44.7$$

* Volume II, "Architectural Construction," Book 1, "Wood Construction," Voss and Varney, John Wiley & Sons, Inc.

† The Building Law of the City of Boston.

‡ The National Board of Fire Underwriters.

Before the design of the slab is undertaken, the arrangement of the beams in a typical panel must be at least temporarily decided. Beams should be and the materials should be estimated for each case, in order to arrive at the most economical and practicable construction to use.

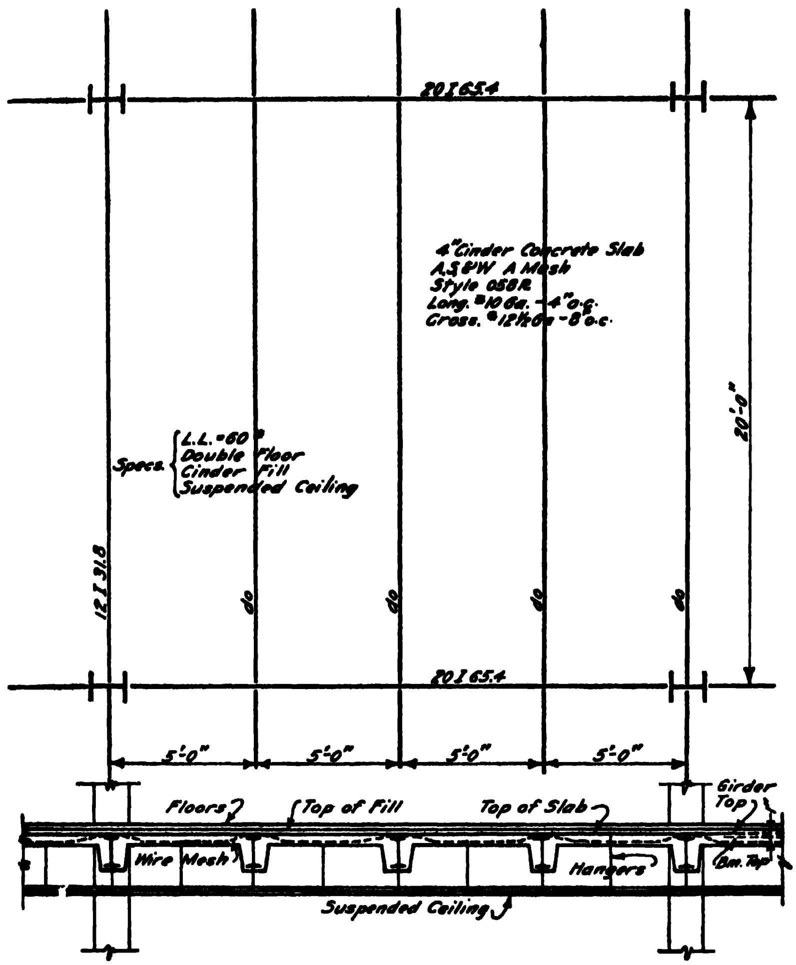


FIG. 229. ONE-WAY CINDER CONCRETE SLAB PANEL, BEAMS AND GIRDERS OF STEEL

used at or near the column center-lines to provide stiffness in the floor construction and to brace the frame. Such beams will also act as temporary bracing until the concrete is cast. The girders which support the floor beams will naturally be placed on the column center-lines and will run in the opposite direction. The panel should be divided into sections of equal length when possible, and should be two, three or four in number, depending upon the span of the girder. A close spacing of the beams gives a smaller slab thickness but may increase the cost of the steel frame. For practical and economical reasons, thin slabs are more expensive and difficult to construct and reinforce, although, theoretically, they require less material. Minimum thicknesses are specified, as already stated, and 6′-0″ spans represent a good average. One or more alternate designs should be made for a typical panel

Illustrative Prob. 142b. Check the sizes of the slab, beams and girder for the typical panel shown in Fig. 229. Assume columns 10″ H sections.

L.L. =	60
1″ Fin. Flr. =	3
1″ Sub. Flr. =	3
2″ Cinder Concrete Fill =	16
4″ Cinder Concrete Slab =	36
Susp. Ceil. =	15
T.L. =	133#/□′

$$M = \frac{w \cdot L^2}{12} = 1.0\, w \cdot L^2 \text{ in.-lbs.}$$

$L = 5'\text{-}0''$

$M = 1.0 \times 133 \times (5)^2$

$= 3300''\#$

$f_c = 300\#/\square''$ $f_s = 20{,}000\#/\square''$

$K = 41.3$ (similar to Table 34)

$$d = \sqrt{\frac{M}{K \cdot b}} = \sqrt{\frac{3300}{41.3 \times 12}} = 2.59''$$

Use 4″ slab
$d = 3.0''$

$$A_s = \frac{M}{f_s \cdot j \cdot d} = \frac{3300}{20{,}000 \times \frac{7}{8} \times 3.0} = 0.059\square''$$

Refer to Table 37. *Use* A. S. & W. triangular mesh
Style 0.58 *R*
Long. wires #10 ga. — 4″ o.c.
Cross wires #12½ ga. — 8″ o.c.

Beams.

$$\text{Load per foot} = 5 \times 133 = 665$$
$$\text{Beam} = 32$$
$$\text{F.P.} = \left(\frac{8 \times 12 - 9.3}{144}\right) \times 108 = 65$$
$$\text{T.L.} = 762\#/\text{ft.}$$

$$M = 1.5 \times 762 \times (20)^2 = 456{,}000''\#$$

$$\frac{I}{c} = \frac{456{,}000}{16{,}000} = 28.6''^3$$

Use 12 I 31.8

$$\frac{I}{c} = 36.0''^3$$

Girders.

$$\text{Beam concentration} = 762 \times 20 = 15{,}240\#$$
$$\text{F.P.}\left(\frac{9.25 \times 20 - 19}{144}\right) \times 108 = 125$$
$$\text{Beam} = 65$$
$$180\#/\text{ft. uniform}$$

End reaction (due to concentrations) = 22,860#
Moment (due to concentrations — see Fig. 229)

$$22.86 \times 10 - 15.24 \times 5 = 152{,}400$$

Moment due to uniform load

$$M = \frac{w \cdot L^2}{8} = \frac{180 \times (20)^2}{8} = 9{,}000$$

$$\text{Moment (total)} = 161{,}400'\#$$

$$\frac{I}{c} = \frac{161{,}400 \times 12}{16{,}000} = 119.0''^3$$

Use 20 I 65.4

$$\frac{I}{c} = 117.0''^3$$

(< 2% overstressed.)

For the limits discussed, the slab thickness will practically always be 4″, but the reinforcement may be varied according to the loads and the slab spans. Consequently, slab tables, such as Table 58, may be used to avoid computations in many instances.

Prob. 142c. Determine the value of K for $f_c = 300\#/\square''$, $f_s = 16{,}000\#/\square''$ and $n = 30$.

Prob. 142d. Check the sizes in the floor frame shown on Pl. 151, Vol. I, the City House. The corresponding architectural plan is shown on Pl. 150 in Vol. I. Refer to Fig. 195, Vol. I, and the accompanying discussion. Cinder concrete slabs.

Prob. 142e. Design a typical interior panel 21′-0″ × 21′-0″ of cinder concrete slabs and steel supporting beams. Use 5′-3″ spacing of beams. L.L. = 75#/□′. Double wood floors 2″ cinder fill, and suspended ceiling. Use $f_c = 300\#/\square''$, $f_s = 18{,}000\#/\square''$ and $n = 30$ for the slabs.

Prob. 142f. Check the sizes shown in Fig. 342 of Vol. I, the Bush Terminal Building. Plates 315 and 320 of that volume show the corresponding architectural plan.

TABLE 58

SAFE LOADS FOR 4″ CINDER CONCRETE SLABS*

Span	T.L. Lbs./Sq. Ft.	Req'd Steel Sq. In. per Ft. Width	Span	T.L. Lbs./Sq. Ft.	Req'd Steel Sq. In. per Ft. Width
4′-0″	86	.0432	5′-6″	86	.0432
	111	.0432		111	.0432
	136	.0432		136	.0528
	161	.0432		161	.0625
	186	.0432		186	.0722
	211	.0433		211	.0818
	236	.0484		236	.0916
	261	.0535		261	.1012
	286	.0587		286	.1110
	311	.0638		311	.1206
	336	.0689		336	.1303
	361	.0741		361	.1400
	386	.0792		386	.1497
	411	.0843		411	.1594
	436	.0895		436	.1691
4′-6″	86	.0432	6′-0″	86	.0432
	111	.0432		111	.0513
	136	.0432		136	.0628
	161	.0432		161	.0744
	186	.0483		186	.0859
	211	.0548		211	.0974
	236	.0613		236	.1089
	261	.0678		261	.1205
	286	.0743		286	.1320
	311	.0808		311	.1436
	336	.0873		336	.1551
	361	.0938		361	.1666
	386	.1003		386	.1782
	411	.1068		411	.1898
	436	.1133		436	.2014
5′-0″	86	.0432	6′-6″	86	.0466
	111	.0432		111	.0601
	136	.0436		136	.0736
	161	.0516		161	.0872
	186	.0597		186	.1007
	211	.0677		211	.1143
	236	.0757		236	.1278
	261	.0837		261	.1414
	286	.0917		286	.1549
	311	.0998		311	.1685
	336	.1078		336	.1820
	361	.1158		361	.1955
	386	.1239		386	.2091
	411	.1319		411	.2226
	436	.1399		436	.2362

* See continuation of table, page 226.

TABLE 38* — *Continued*

Span	T.L. Lbs./Sq. Ft.	Req'd Steel Sq. In. per Ft. Width	Span	T.L. Lbs./Sq. Ft.	Req'd Steel Sq. In. per Ft. Width
7'-0"	86	.0540	8'-0"	86	.0706
	111	.0697		111	.0911
	136	.0855		136	.1116
	161	.1011		161	.1321
	186	.1168		186	.1526
	211	.1325		211	.1732
	236	.1481		236	.1936
	261	.1639		261	.2142
	286	.1796		286	.2346
	311	.1953		311	.2553
	336	.2110		336	.2758
	361	.2267		361	.2963
	386	.2424		386	.3168
	411	.2581		411	.3374
	436	.2738		436	.3579
7'-6"	86	.0620			
	111	.0800			
	136	.0981			
	161	.1160			
	186	.1341			
	211	.1522			
	236	.1701			
	261	.1882			
	286	.2061			
	311	.2243			
	336	.2423			
	361	.2603			
	386	.2784			
	411	.2964			
	436	.3145			

* Table based upon $f_c = 300\#/\square''$, and $f_s = 20{,}000\#/\square''$. For f_s = other values, i.e., 16,000#/□" or 18,000#/□", areas of steel may be obtained by inverse proportion.

For spans greater than 8'-0", use stone concrete slabs.

Based on $M = \frac{w \cdot L^2}{10}$. For $\frac{w \cdot L^2}{12}$, use values corresponding to spans 10% less. For $\frac{w \cdot L^2}{8}$ use values corresponding to spans 25% greater.

143. Variations From the Typical Construction.

In special instances, the position of the slab with respect to the depth of the floor beams may be varied. In the typical section shown in Fig. 230 the plaster may be applied directly to the soffit of the slabs and to the beam protection, making a paneled ceiling, or this effect may be increased by introducing false beams of light furring covered with wire lath and plaster. In the typical construction, the mesh is supported on the tops of the beams and is often allowed to sag to the proper distance from the bottom of the slab, usually specified to be not less than $\frac{5}{8}''$. An objection to the use of wire reinforcement so placed is the danger of the mesh sagging out of position. This situation is improved by aligning the mesh to raising rods as in Fig. 230 (*h*).

To overcome these objections, an alternate method of construction is to use a heavier and stiffer form of reinforcement, which is called **permanent centering,** such as Hy-rib, T-rib chanelath, and self-sentering, as shown in Fig. 230 (*a*) and (*b*). The center of gravity of such reinforcement is about 0.3" above the bottom of the mesh, and this feature should be taken into account in the calculations for slab depths. Sectional areas of such reinforcement are given in Table 38. Considerable form work is eliminated by using this reinforcement. In (*a*), the centering is supported by the beam side forms, and in (*b*) by the terra cotta fire-protection blocks, which incidentally eliminate the forms for any beam protection of concrete. An objection to such methods is that it is difficult to obtain sufficient fire-protection underneath the centering. For spans greater than 5'-0", it should be partially

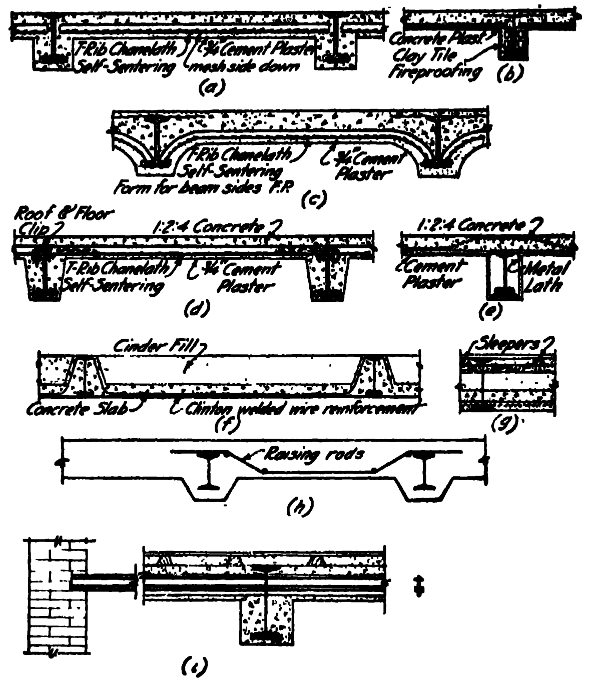


FIG. 230

supported at mid-span by runner bars parallel to the beams (usually $\frac{3}{4}''$ channels) suspended by wires 12" to 16" o.c., to prevent excessive sagging. Figure 230(*c*) shows another form of construction to eliminate beam-side forms. It avoids the sharp junctions of the slabs and beams and is satisfactory if the resulting ceiling effect is not objectionable. Another method of supporting the permanent centering is shown in Fig. 230 (*d*), in which it rests directly on top of the I-beams. This provides additional headroom between the beams. For heavy loads or long spans, wire mesh should be added in the tops of the slabs over the I-beams, as shown in (*e*), to provide for the negative moment at the supports. Figure 230 (*f*) shows a form of flush ceiling construction, and (*g*) a variation of the same with permanent centering. An objection to such construction is the thick and unnecessarily heavy cinder fill required. There are also several patented systems on the market for reinforcing concrete slabs. Figure (*i*) shows the Columbian patented construction. Except for the features noted, the design of the foregoing slabs is similar to that already discussed (Art. 142).

Illustrative Prob. 143a. Design a slab to span 6'-0", using the construction indicated in Fig. 230 (*a*). L.L. = 60#/□'. Use 1 : 2 : 5 cinder concrete.

L.L.	=	60
⅞" Fin. Flr.	=	3
⅞" Sub Flr.	=	3
2" Cinder Fill	=	16
4" Slab	=	36
Plaster	=	8
T.L.	=	126#/□'

f_c = 300#/□", f_s = 18,000#/□"

$n = 30, \quad K = 48$

$$M = 1.2\,w \cdot L^2 = 1.2 \times 126 \times (6)^2 = 5430''\#$$

$$d = \sqrt{\frac{5430}{48 \times 12}} = 3.07''$$

Use 4" slab (d = 3.5)

$$A_s = \frac{5430}{18{,}000 \times \frac{7}{8} \times 3.5} = 0.09\square''$$

(Table 38)

Use #28 ga. T-rib chanelath

Prob. 143b. Design a typical interior panel 20'-0" × 20'-0" for the construction indicated in Fig. 230 (*f*). Use 5'-0" spacing of beams. Assume 10" cinder fill.

SECTION 10B

STONE CONCRETE SLABS

144. Typical Construction.

Floor construction with stone concrete slabs and steel supporting beams is in many ways similar to that when cinder concrete is employed. Figure 231 shows a section of a typical floor. Floor beams should be located on the column center-lines for the reasons given in Art. 142. The intermediate beams are usually spaced farther apart than for cinder concrete slabs, as those of stone concrete are stronger. Since the minimum thickness of floor slabs is generally specified as 4", the beams should be spaced far enough apart to develop the maximum efficiency of such a slab, at least.

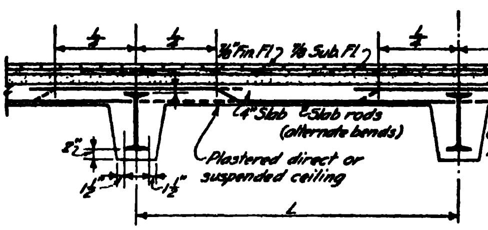


FIG. 231

Illustrative Prob. 144a. What is the maximum span for a 4" stone concrete slab for a L.L. of 60#/□', typical construction? Use 2000# concrete.

L.L.	=	60
Fin. Flr.	=	3
Sub Flr.	=	3
2" Cinder Fill	=	16
4" Stone Concr. Slab	=	50
Pl. Ceiling	=	5
T.L.	=	137#/□'

$$M = 1.2\,w \cdot L^2 = 1.2 \times 137 \times L^2$$

f_c = 650#/□"

f_s = 16,000#/□"

$n = 15, \quad K = 107.4$

$M_r = K \cdot b \cdot d^2$

For 4" slab, $d = 3.0''$

$$107.4 \times 12 \times (3.0)^2 = 1.2 \times 137 \times L^2$$

$$L = 7.04'$$

Use 7'-0" span

From the above illustration, it is seen that the spans may be larger than those for cinder concrete slabs, of course depending upon the loads. For this reason, small rods are more commonly used for reinforcement in such construction, as the required areas of steel often exceed the values available with mesh reinforcement. However, for light loads and relatively short spans, mesh reinforcement may be used, as discussed in Art. 106, or the relation of the slab to the supporting beam may be varied, as considered in Art. 143.

The usual spacing of beams is from 6'-0" to 8'-0". One-way slabs spanning all the way between girders on the column center-lines in one direction is heavy and expensive construction, and is seldom resorted to unless extremely close conditions of headroom control. The thicker slab would be an advantage in resisting accidental, concentrated loads, but it is relatively expensive. For slabs and steel beams with girders in one direction only, the dead load is not materially in excess of concrete joist construction. The former provides better resistance to concentrated loads than the thin slab between the joists. The heavier reinforcement in the joist-floor allows easier placing of steel. Less headroom is required in the latter type (under the beams) if a flat ceiling is desired, although paneled types of ceilings may be worked out with the steel cross beams as a basis in the slab type. A disadvantage of the slab and beam floor is that no standard forms, similar to joist construction, are available.

Illustrative Prob. 144b. Calculate the required sizes for a L.L. of 60#/□' and a 20'-0" × 20'-0" panel, as indicated by the typical engineer's sketch in Fig. 232. Use Boston Law.

L.L.	=	60
Fin. Flr.	=	3
Sub Flr.	=	3
2" Cinder Fill	=	16
4" Slab	=	50
Pl. Ceiling	=	5
T.L.	=	137#/□'

Typical span fully continuous

Use $M = \frac{w \cdot L^2}{12} = 1.0\,w \cdot L^2 ''\#$

f_c = 0.325 × 2200 = 715#/□"

f_s = 18,000#/□", $n = 15$

$K = 125.$

Use one intermediate beam, slab span, L = 10'-0"

$$M = 1.0 \times 137 \times (10)^2 = 13{,}700''\#$$

$$d = \sqrt{\frac{M}{K \cdot b}} = \sqrt{\frac{13{,}700}{125 \times 12}} = 3.02''$$

Use 4" Slab (d = 3.0")

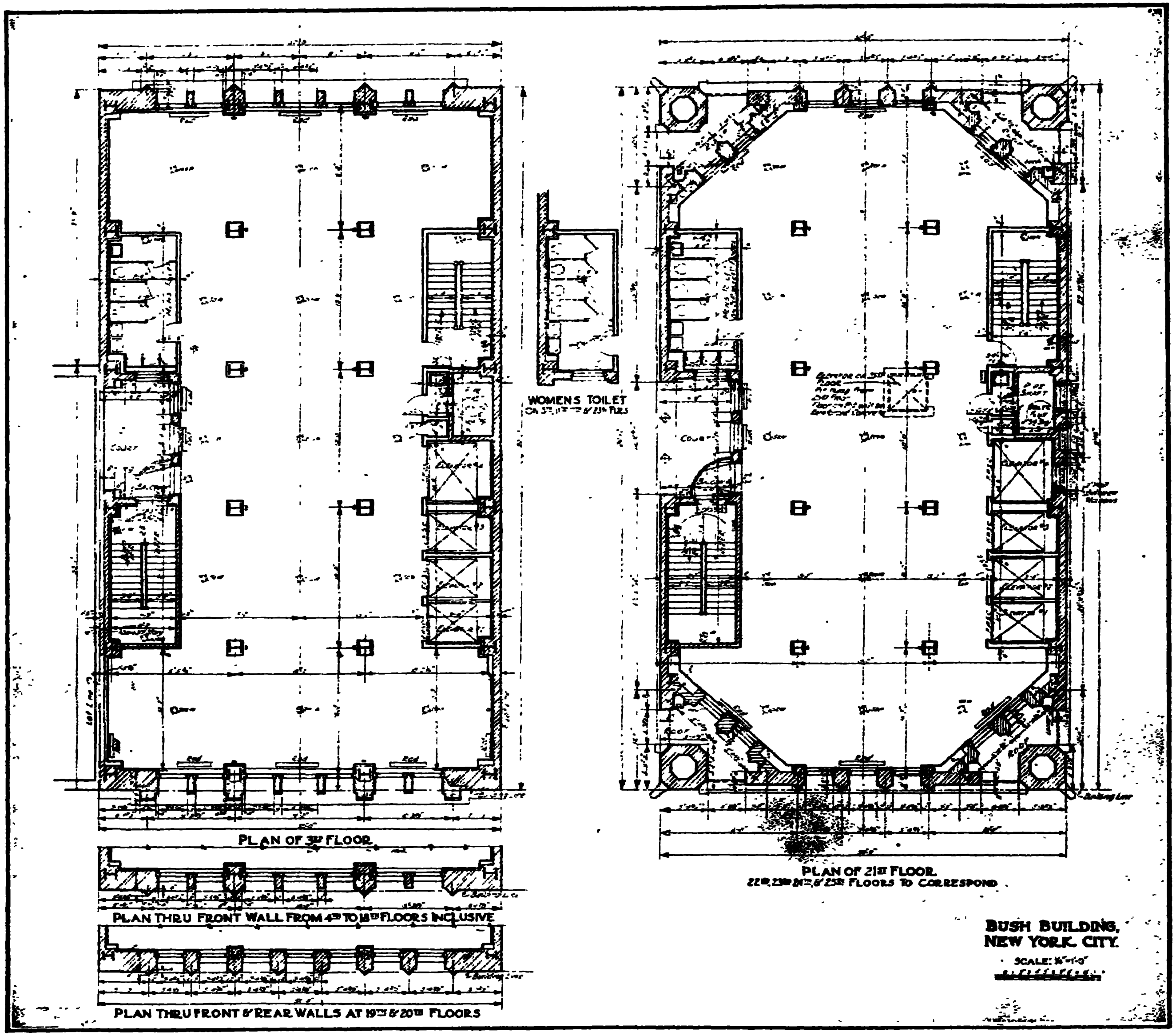


PLATE 20 THE OFFICE BUILDING
TYPICAL FLOOR PLANS
HELMLE & CORBETT, ARCHITECTS

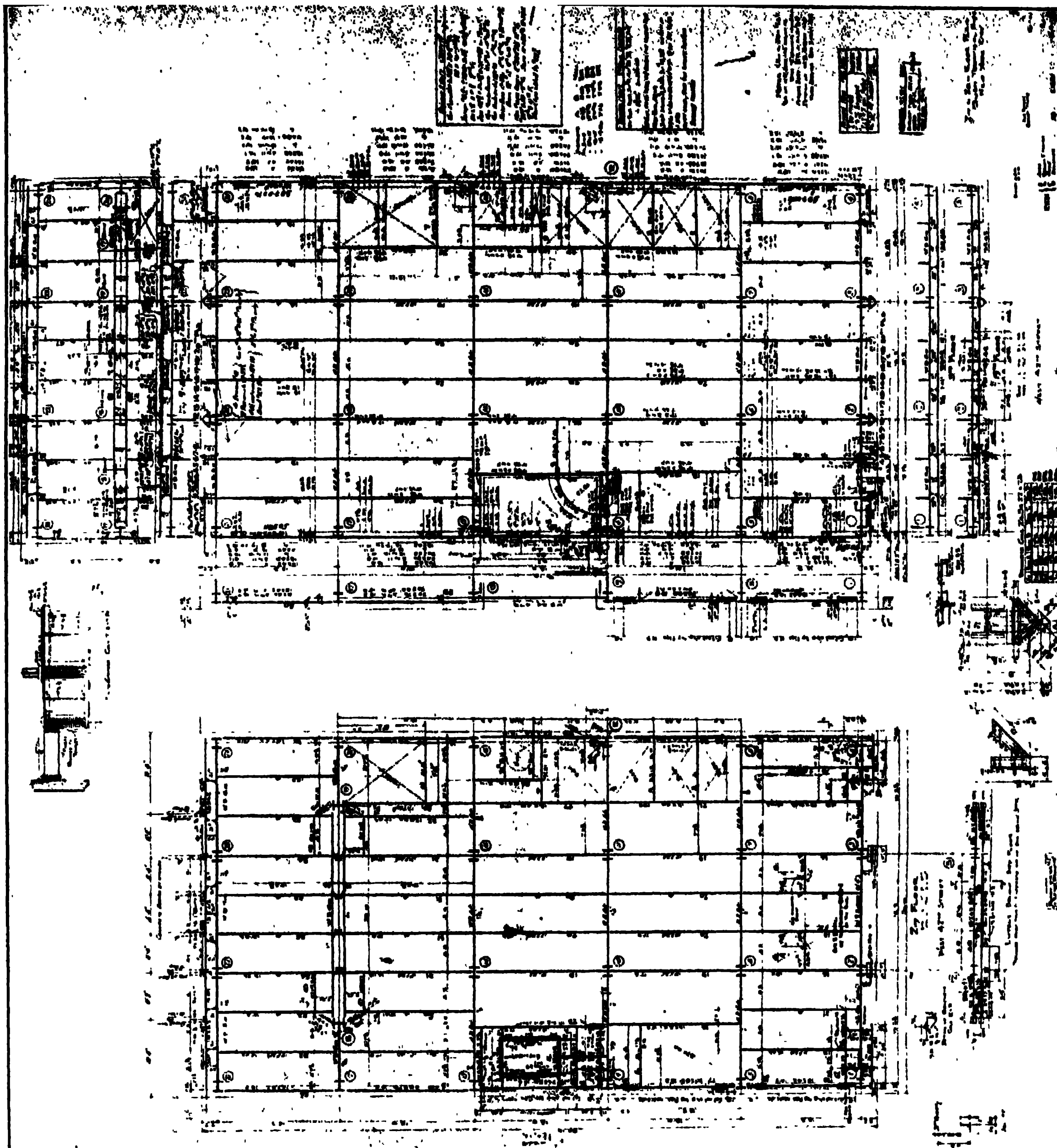


PLATE 21 THE OFFICE BUILDING
TYPICAL FLOOR FRAMING PLANS
HELMLE & CORBETT, ARCHITECTS

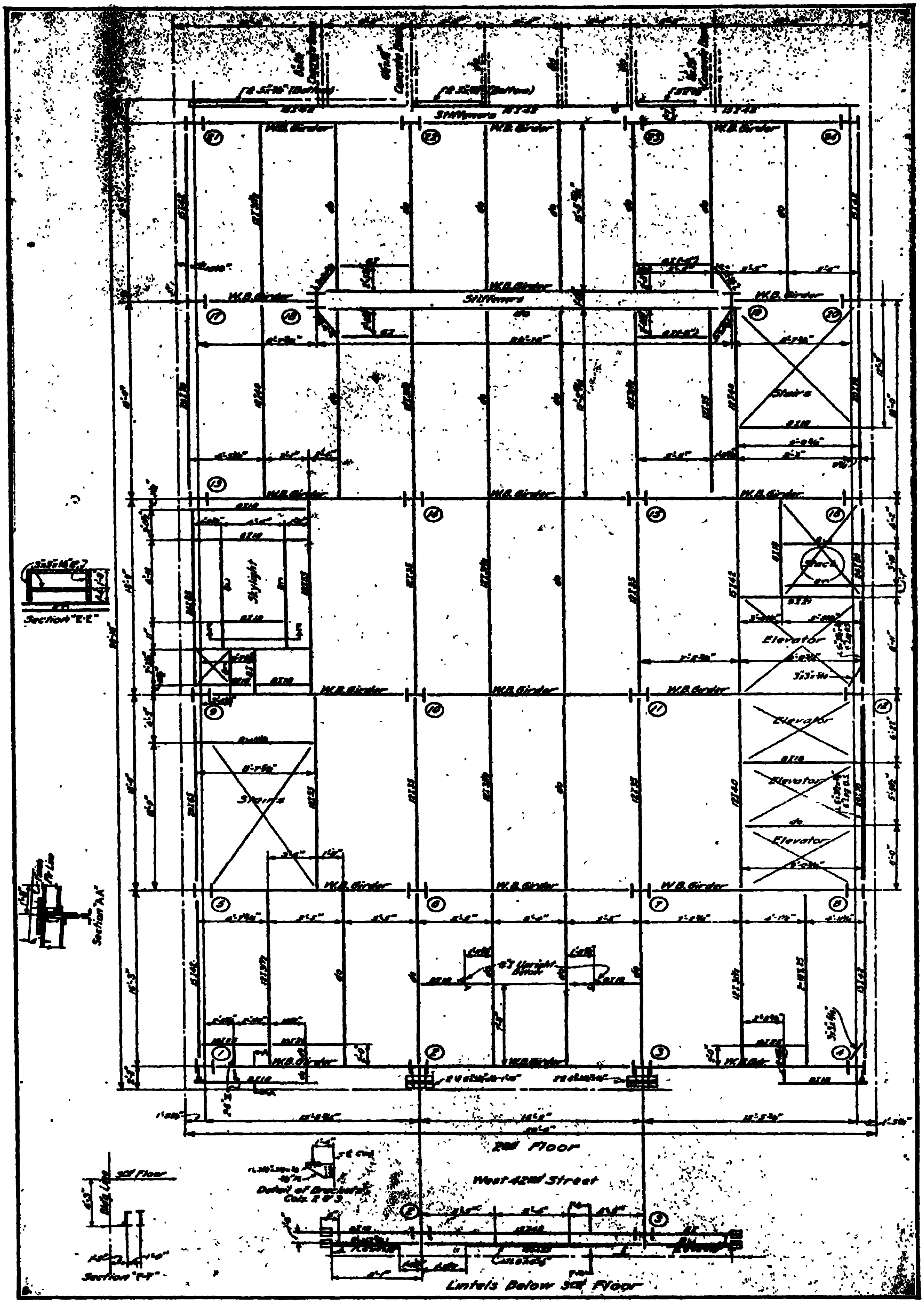


PLATE 22 ENGINEERS LAYOUT FOR STEEL FRAME 230

$$A_s = \frac{M}{f_s \cdot j \cdot d} = \frac{13,700}{18,000 \times \frac{7}{8} \times 3.0} = 0.29\square''$$

$$\tfrac{3}{8}''\,\phi = 0.11\square'' \quad \frac{0.29}{0.11} = 2.63 \quad \frac{12}{2.63} = 4.5$$

$$\tfrac{1}{2}''\,\phi = 0.196 \quad \frac{0.29}{0.196} = 1.48 \quad \frac{12}{1.48} = 8.1$$

Use $\frac{1}{2}''\phi - 8''$ *o.c.*
Bend up every other rod

Load on beam = 10 × 137 = 1370#/ft.
Beam and haunch = 190 (see Prob. 140*a*)
T.L. = 1560#/ft.

$$M = 1.5\,w \cdot L^2 = 1.5 \times 1560 \times (20)^2 = 936,000''\#$$

$$\frac{I}{c} = \frac{M}{s} = \frac{936,000}{16,000} = 58.4''^3$$

Use 15 I 42.9

The computations for the reinforcement required for slabs may often be minimized by the use of slab tables, such as Table 59, if the conditions surrounding the design correspond with the table. The rods which are used for slab reinforcement are usually bent at the quarter-points of the span and extended to the quarter-point of the adjoining span, to provide for negative moment. That is, a rod is bent at one end to the top of the slab and remains straight at the other end in the bottom of the slab, as illustrated in the section in Fig. 232, so that every other rod at a support is bent up, and all rods are bent at one end only. Since the same is done for each panel, there are as many top rods at the supports as there are bottom rods at mid-span. When the negative moment is larger than the positive moment, as is occasionally the case, extra straight rods should be added over the support to make up the difference

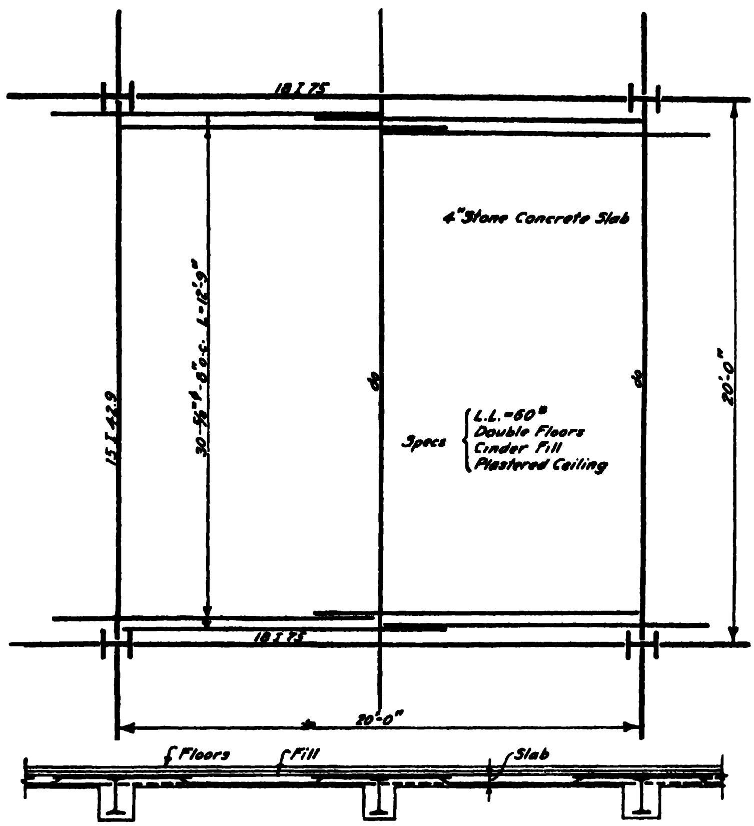


Fig. 232. One-Way Stone Concrete Slab Panel, Beams and Girders of Steel

$$R_1 = \frac{w \cdot L}{2} = \frac{1560 \times 20}{2} = 15,600\#$$

Concentration on girder = 31,200#

$$M = \frac{P \cdot L}{4} = \frac{31,200 \times 20}{4} = 156,000'\# = 1,870,000''\#$$

Beam and haunch = 240#/ft.

$$M = 1.5\,w \cdot L^2 = 1.5 \times 240 \times (20)^2 = 144,000$$

Total M = 2,014,000″#

$$\frac{I}{c} = \frac{2,014,000}{16,000} = 125.6''^3$$

Use 18 I 75

Prob. 144c. Design a typical interior panel 18′-0″ × 18′-0″ of stone concrete slab and steel beam construction for a L.L. of 150#/□′. Use a 9′-0″ spacing of beams. 1″ granolithic finish floor, no fill. Use f_c = 650#/□″ and f_s = 18,000#/□″ for the slabs.

TABLE 59

SAFE LOADS FOR 4" STONE CONCRETE SLABS*

Span	T.L. Lbs./Sq. Ft.	Req'd Steel, Sq. In. per Ft. Width	Span	T.L. Lbs./Sq. Ft.	Req'd Steel, Sq. In. per Ft. Width	Span	T.L. Lbs./Sq. Ft.	Req'd Steel, Sq. In. per Ft. Width
4'-0"	100	.0432	5'-6"	100	.0432	7'-0"	100	.0545
	125	.0432		125	.0432		125	.0681
	150	.0432		150	.0504		150	.0817
	175	.0432		175	.0588		175	.0953
	200	.0432		200	.0673		200	.1089
	225	.0432		225	.0756		225	.1225
	250	.0445		250	.0840		250	.1361
	275	.0480		275	.0924		275	.1497
	300	.0534		300	.1008		300	.1633
	325	.0578		325	.1092		325	.1770
	350	.0622		350	.1176		350	.1906
	375	.0666		375	.1260		375	.2042
	400	.0711		400	.1344		400	.2178
	425	.0755		425	.1428		425	.2314
	450	.0800		450	.1512		450	.2450
4'-6"	100	.0432	6'-0"	100	.0432	7'-6"	100	.0625
	125	.0432		125	.0500		125	.0782
	150	.0432		150	.0600		150	.0938
	175	.0432		175	.0700		175	.1094
	200	.0450		200	.0800		200	.1250
	225	.0506		225	.0900		225	.1407
	250	.0563		250	.1000		250	.1563
	275	.0618		275	.1100		275	.1719
	300	.0675		300	.1200		300	.1875
	325	.0731		325	.1300		325	.2032
	350	.0787		350	.1400		350	.2188
	375	.0843		375	.1500		375	.2345
	400	.0900		400	.1600		400	.2500
	425	.0956		425	.1700		425	.2657
	450	.1012		450	.1800		450	.2814
5'-0"	100	.0432	6'-6"	100	.0470	8'-0"	100	.0711
	125	.0432		125	.0587		125	.0889
	150	.0432		150	.0705		150	.1067
	175	.0486		175	.0822		175	.1244
	200	.0556		200	.0940		200	.1422
	225	.0625		225	.1056		225	.1600
	250	.0695		250	.1174		250	.1778
	275	.0764		275	.1291		275	.1956
	300	.0834		300	.1408		300	.2133
	325	.0903		325	.1526		325	.2311
	350	.0972		350	.1643		350	.2489
	375	.1042		375	.1761		375	.2667
	400	.1111		400	.1878		400	.2845
	425	.1181		425	.1995		425	.3022
	450	.1250		450	.2113		450	.3200

* Based on f_c = 650#/□" and f_s = 20,000#/□". For other values of f_s, such as 16,000#/□", and 18,000#/□", area of steel may be obtained by inverse proportion.

For $M = \frac{w \cdot L^2}{12}$, use data for spans 10% greater than listed. For $M = \frac{w \cdot L^2}{8}$, use data for spans 25% less than tabulated.

Based upon $M = \frac{w \cdot L^2}{10}$.

Weight of slab deducted.

145. Two-Way Slabs and Steel Girders.

An alternate type of construction employing concrete slabs and a steel floor frame is the use of structural steel girders on the column center lines in both directions, supporting a stone concrete slab reinforced in both directions, as illustrated in Fig. 233. The girders in each direction help to brace the columns and to stiffen the frame. This type of floor is not as common as the use of intermediate beams and slabs, but when the panels are square, or nearly so, some economy in the slab design may be effected. The design and limitations of slabs reinforced in two directions is discussed in Art. 107.

The **load brought to the girders** by two-way slabs is **not uniformly distributed** and the moment and shear calculations for such members involve special considerations. It is consistent to assume that the load on a panel is distributed to the supporting girders in the same ratio that the slab was designed for, in the two respective directions. The ratio carried in the transverse direction of the slab is expressed by

$$r = \frac{L_L}{L_B} - 0.5 \quad \text{(Art. 107)},$$

and the remainder is carried in the longitudinal direction of the slab. Thus if a 12′-0″ × 14′-0″ panel carries a total load of 240#/□′ and is reinforced in both directions, the following girder loads result:

$$L_L = 14'\text{-}0'' \qquad L_B = 12'\text{-}0''$$

$$r = \frac{14}{12} - 0.5 = 0.66$$

Total load on 14′-0″ girder from one panel

$$= \frac{0.66 \times 240 \times 14 \times 12}{2} = 13{,}400\#$$

Total load on 12′-0″ girder from one panel

$$= \frac{0.34 \times 240 \times 14 \times 12}{2} = 6{,}700\#.$$

If a girder is an intermediate one, the load is of course that from two panels. For square panels, the load from one panel upon a girder is obviously one-quarter of the panel load.

The distribution of the load on a particular girder is not uniform and varies according to the ordinates of a parabola.

SPECIFICATION CLAUSE*

Loads on Beams from Two-way Slabs Beams supporting rectangular slabs reinforced in both directions shall be assumed to take the proportions of load as determined by the formula in this section ($r = \frac{l}{b} - 0.5$), the distribution of the load being assumed to vary in accordance with the ordinates of a parabola having its vertex at mid-span.

* The Building Law of the City of Boston.

Such a variation is represented in Fig. 233 (*a*). The maximum moment for a simply supported beam may be expressed as follows:

$$M_{max} = \frac{W}{2} \times \frac{L}{2} - \frac{W}{2} \times \frac{3L}{16} = \frac{5W \cdot L}{32} \qquad (1)$$

in which W is calculated as illustrated above. Some engineers assume a triangular distribution of load, as illustrated in Fig. 233 (*b*), in order to avoid the calculations based upon the assumption of parabolic distribution. The moment in such a case may be expressed as

$$M_{max} = \frac{W}{2} \times \frac{L}{2} - \frac{W}{2} \times \frac{L}{6} = \frac{W \cdot L}{6} \qquad (2)$$

for a simply supported beam. By comparing formulas (1) and (2), it is seen that the second is slightly larger and is on the safe side, as

$$\frac{5}{32} = 0.156 \quad \text{and} \quad \frac{1}{6} = 0.167.$$

Illustrative Prob. 145a. Design a typical interior panel 20′-0″ × 20′-0″ for a L.L. of 60#/□′, using a two-way slab construction.

L.L.	= 60	$r = \frac{L_L}{L_B} - 0.5 = 1.0 - 0.5 = 0.5$
⅞″ Fin. Flr.	= 3	
⅞″ Sub Flr.	= 3	(Square panel)
2″ Cinder Fill	= 16	Basic moment, $M = \frac{w \cdot L^2}{12}$′#
5″ Slab	= 63	
Plaster Ceil.	= 5	Fully continuous
T.L.	= 150#/□′	

$$M = 0.5 \times 1.0 \times 150 \times (20)^2 = 30{,}000''\#$$

$$d = \sqrt{\frac{30{,}000}{12 \times 117}} = 4.5 \qquad \textit{Use 6″ Slab}$$

Assume ½″ϕ rods
d (upper layer) = 4.5

$$A_s = \frac{30{,}000}{18{,}000 \times \frac{7}{8} \times 4.5}$$

$A_s = 0.42''$ Use ⅜″ϕ rods — 6″ o.c. middle half of panel, and ⅜″ϕ — 12″o.c. outer quarters, both ways.

Total floor load on girder, ¼ × 20 × 20 × 150 = 15,000#
× 2
= 30,000

Beam stem $\frac{10 \times 12}{144} \times 150 \times 20$ = 2,500

W = 32,500#

$$M_{max} = \frac{5W \cdot L}{32}\text{′\#} = \frac{5 \times 32{,}500 \times 20}{32} = 101{,}300'\#$$

$$\frac{I}{c} = \frac{101{,}300 \times 12}{16{,}000} = 76.1''^3$$

Use 18 I 54.7

Figure 233 shows a design sketch.

Illustrative Prob. 145b. Design a typical interior panel 16′-0″ × 18′-0″ for a L.L. of 150#/□′, using a two-way slab construction.

L.L. = 150	
1″ Grano. = 12	$r = \dfrac{L_L}{L_B} - 0.5 = \dfrac{18}{16} - 0.5 = 0.62$
8″ Slab = 100	
Pl. Ceiling = 5	Basic moment $= \dfrac{w \cdot l^2}{12}$
T.L. = 267#/□′	Fully continuous

Load on 18′-0″ girder

$$2 \times \frac{0.62 \times 267 \times 16 \times 18}{2} = 47{,}500$$

$$\text{Beam stem } \frac{12 \times 13}{144} \times 150 \times 18 = 2{,}900$$

$$W = 50{,}400\#$$

Load on 16′-0″ girder

$$\frac{2 \times 0.38 \times 267 \times 16 \times 18}{2} = 29{,}200$$

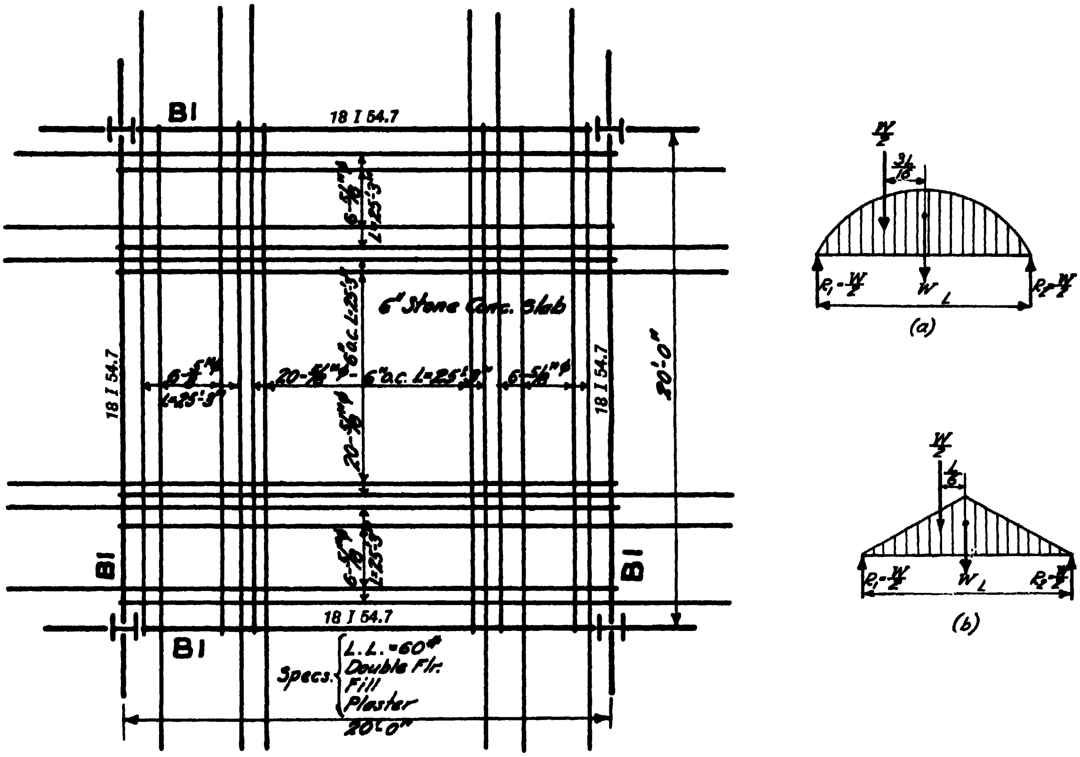


Fig. 233

$M = 0.62 \times 1.0 \times 267 \times (16)^2 = 42{,}400''\#$
(Transverse direction)

$$d = \sqrt{\frac{42{,}400}{117 \times 12}} = 5.5 \qquad \textit{Use 7'' slab}$$

$$A_s = \frac{42{,}400}{18{,}000 \times \frac{7}{8} \times 6} = 0.45\square''$$

Use $\frac{3}{8}''\phi$ — 6″ o.c. middle half panel
$\frac{3}{8}''\phi$ — 12″ o.c. outside quarters — transverse

$M = 0.38 \times 1.0 \times 267 \times (18)^2 = 32{,}800''\#$
(Longitudinal direction)

$$A_s = \frac{32{,}800}{18{,}000 \times \frac{7}{8} \times 5.38} = 0.386\square''$$

$d = 6.0 - 0.62 = 5.38$ for upper layer

Use $\frac{3}{8}''\phi$ — 9″ o.c. middle half panel
$\frac{3}{8}''\phi$ — 18″ o.c. outside quarters — longitudinal

$$\text{Beam stem } \frac{10 \times 11}{144} \times 150 \times 16 = 1{,}900$$

$$W = 31{,}100\#$$

The design of the girders is similar to that of Illustrative Prob. 145a.

Prob. 145c. Design a typical interior panel 18′-0″ × 18′-0″ for a L.L. of 100#/□′, using a two-way slab. Wood floor.

Prob. 145d. Determine the sizes of the members and the necessary reinforcement for a typical interior panel 12′-0″ × 14′-0″ for a L.L. of 60#/□″. Use two-way slab. 1″ granolithic finish floor.

Prob. 145e. What are the values of the moments for the design of the slab in Prob. 145*d* if the panel under consideration is an end panel in the 12′-0″ direction and a mid-panel in the 14′-0″ direction? What are the values for a corner panel?

146. Sectional Systems.

Several patented systems have been on the market which involve the use of pre-cast sections. These are placed between steel beams instead of cast-in-

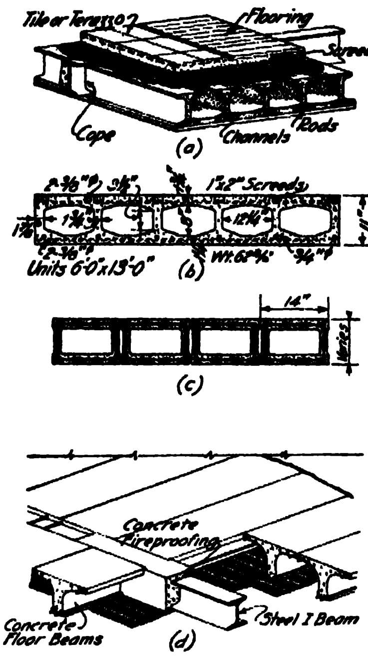


FIG. 234

place slabs. The sections are made of a fairly rich mix of concrete with small sizes of aggregates, and are reinforced with steel rods or other metal. Disadvantages of such construction are that breakage is a considerable item during the transportation and erection, relatively high costs are often the case, and it is necessary to use a uniform spacing of floor beams, which is not always practicable. Figure 234 (*a*), (*b*), (*c*), and (*d*) shows the Waite, Siegwart, Climax, and Watson systems, respectively. They are most adaptable for standardized buildings.

147. Steel Form Floors.

An alternate form of floor construction which is used occasionally with a steel frame is that in which light steel members are used to carry the load between the floor beams, combined with a light concrete to act as a fill. Plain steel plates, patented

FIG. 235

types, such as "Buckeye" or "Multiplex,"* or various kinds of **trough plates** have been used for such work. Some of the latter are illustrated in Fig. 235. Such types of floors have been confined principally to steel mill buildings and the like.

148. Segmental Arches.

Segmental arches of concrete, sprung between steel beams, is another alternate, though uncommon, type of floor (Art. 139).

SECTION 10c

GYPSUM SLABS

149. Pre-Cast Construction.

A form of construction which is adapted for light loads, and which has been quite recently placed upon the market, is that of using factory moulded units of gypsum supported by standard rolled structural steel channels, spaced 30″ o.c., spanning between structural steel girders.† Figure 236 shows a perspective view of this construction.

The slabs are made of high grade calcined gypsum and a small percentage of fine wood planer chips, which act as a binder. Water is added and the whole automatically mixed, and then cast into steel molds. When dry, these are shipped to the job for erection. The ceiling slabs are all one standard size, 30″ × 24″ × 2″ and are reinforced by two $\frac{3}{8}$″ × $\frac{1}{4}$″ flat steel bars placed on edge. These span from back to back of channels, and are supported by $\frac{1}{8}$″ × 1″ flat steel hangers clamped over the flanges of the channels. The ends of the reinforcing bars in the ceiling slabs project about 1″ beyond the ends of the slab and are made to pass through a slot cut in the ends of the hangers. The ends of the reinforcing bars, as well as the ends of the hangers, fit into pockets left in the slabs. The floor slabs are set after the ceiling slabs are in place. These are also of one standard size, 30″ × 24″ × $2\frac{1}{2}$″, and are reinforced with cold-drawn steel wire rods $\frac{3}{16}$″ ϕ — 6″ o.c. These rods extend $2\frac{1}{2}$″ beyond each end of each unit at a plane about $\frac{3}{4}$″ below the top of the slab. The ends of the rods are caught together and twisted by a mechanical device and hammered down into pockets left in the slabs to

* For illustrations and section-moduli of various types of such plates, see M. S. Ketchum's "Structural Engineers' Handbook," McGraw-Hill Publishing Co.

† Called Pre-Cast Gypsteel Floor Construction, patented and furnished by the Structural Gypsum Corporation, New York City.

receive them. Both floor and ceiling slabs are rabbeted on the edges. The slots thus formed, and the pockets previously mentioned, are filled with gypsum grout.

The flanges of girders which project below the ceiling are given protection by soffit slabs, either 2″ or 3″ thick (depending upon building code requirements), held in place by $\frac{3}{4}''\times\frac{1}{8}''$ steel straps attached to the upper flanges. These slabs are reinforced in a manner similar to the other slabs, and extend $1\frac{1}{2}''$ beyond the toes of the girder flange on each side. Projecting webs of the girders are protected by special haunch slabs or by casting in faces of gypsum composition. A 2″ cinder concrete fill is placed on top of the slabs and the finish flooring is placed in the usual way.

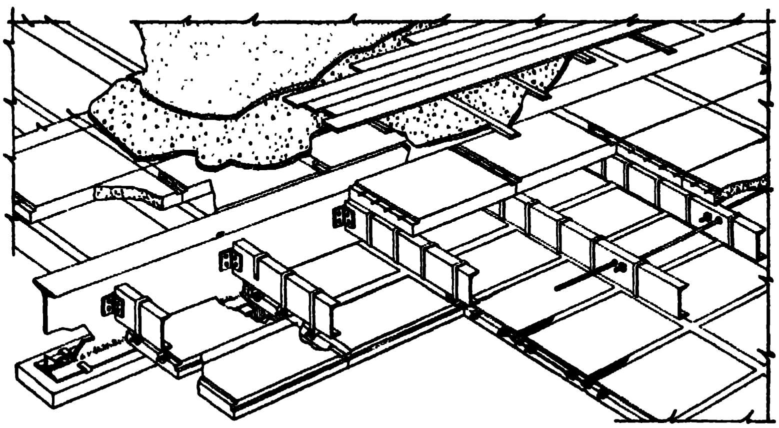


Fig. 236

Certain precautions should be taken with respect to the structural steel supporting frame. If convenient, the column spacings in a direction parallel with the girders should be in multiples of 2′-6″. This means that tie-beams for the columns can then be employed without complicating the framing. If special beams are required for one reason or another in a direction parallel to the channels, they are not counted upon as a part of the floor construction. Section E–E in Fig. 237 shows a detail of this kind. The spacings of 2′-6″ for the channels should be worked out so that any odd amounts (less than 2′-6″) occur adjacent to the sides of the panel. The slabs can be cut in the field for such conditions. The channels may be placed $1\frac{1}{2}''$ or 2″ below the tops of the girders in order to eliminate coping. This also reduces the amount of fireproofing necessary for the girders. If the flanges of the floor channels are less than $2\frac{1}{4}''$ wide, it is necessary to use special temporary bearing plates to support the slabs until they are tied and grouted into place.* Tie-rods (about 5′-0″ o.c.) should be used to give the channels lateral support (Art. 11). In a given panel, the tops of all the supporting channels must be level, and these members must of course all be of the same depth. Figure 237 shows common details for the floor construction, and Fig. 238 illustrates special details often required, and Fig. 239 shows a typical installation.

The strength of the floor slabs, based upon supports 2′-6″ o.c., has been shown by tests to be safe for loads up to 150#/□′, showing a high factor of safety. For purposes of design computations, the following weights are convenient:

Steel channels (average)	5#/□′
$2\frac{1}{2}''$ floor slabs	12
2″ ceiling slabs	9
2″ cinder fill	16
2″ soffit blocks	9†
3″ soffit blocks	14†
2″ haunch blocks	9†
Finish flooring	(See Table 30)

Some of the advantages claimed for this type of construction are:

(1) Form work is eliminated, thus expediting the erection and allowing free space under the floor,

(2) No water is necessary (except for negligible amounts for grouting), thus eliminating time required for setting and drying out, as well as inconvenience in the story below,

(3) A low dead load of floor construction is possible because of the light weight of the gypsum slabs,

(4) The construction is favorably resistant to sound transmission, and the heat conductance is low,

(5) The erection can be carried on at temperatures when there might be danger from freezing in other forms of construction.

Some engineers do not consider that the construction is as fire-resisting as that involving concrete slabs and protected steel, however.

* The floor slabs span from center to center of channel flanges, and a bearing of at least $1\frac{1}{4}''$ is necessary for each slab. The temporary bearing plates are furnished with the slabs in such cases.

† Calculated for surface areas of girder projections only.

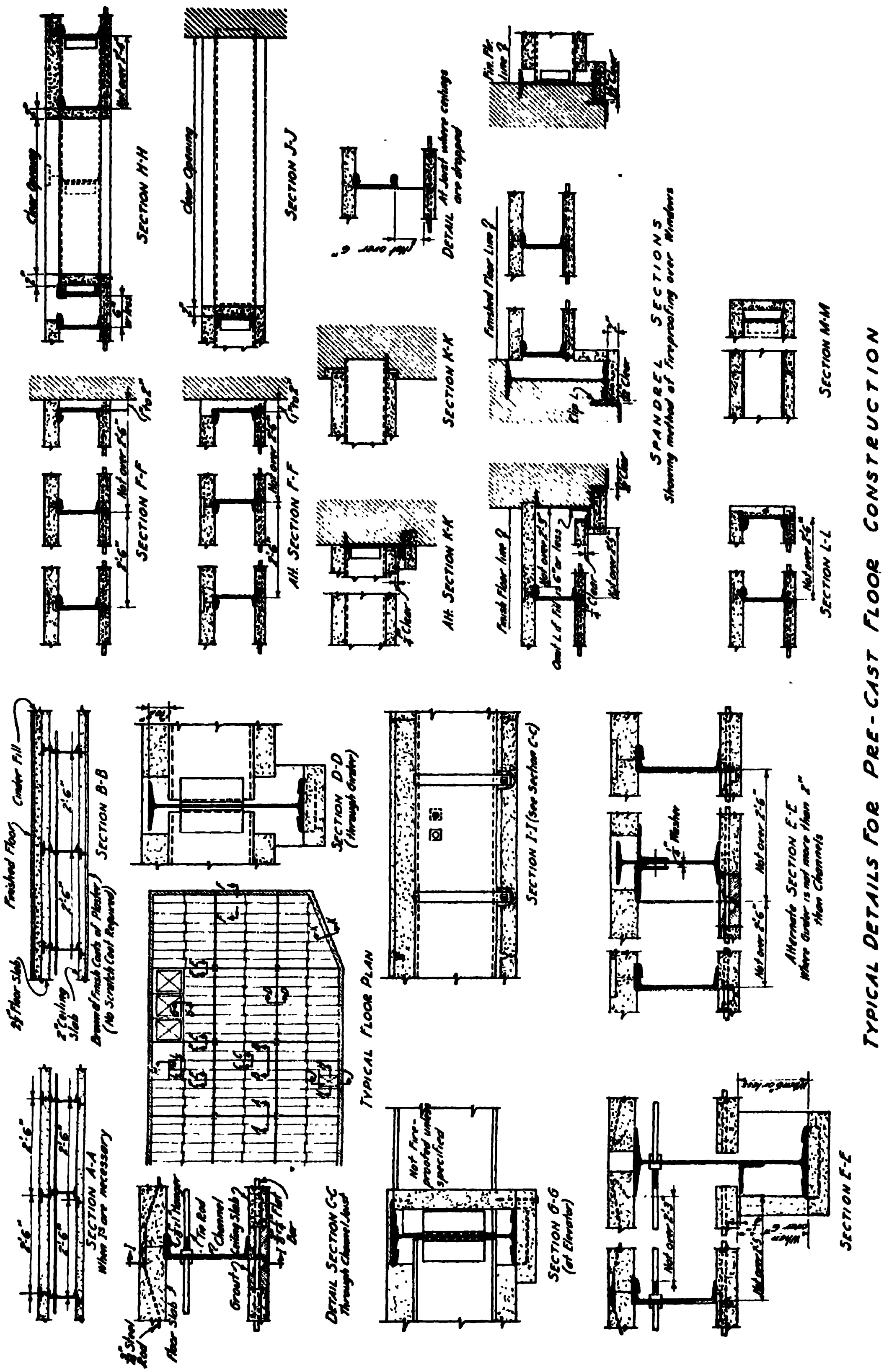
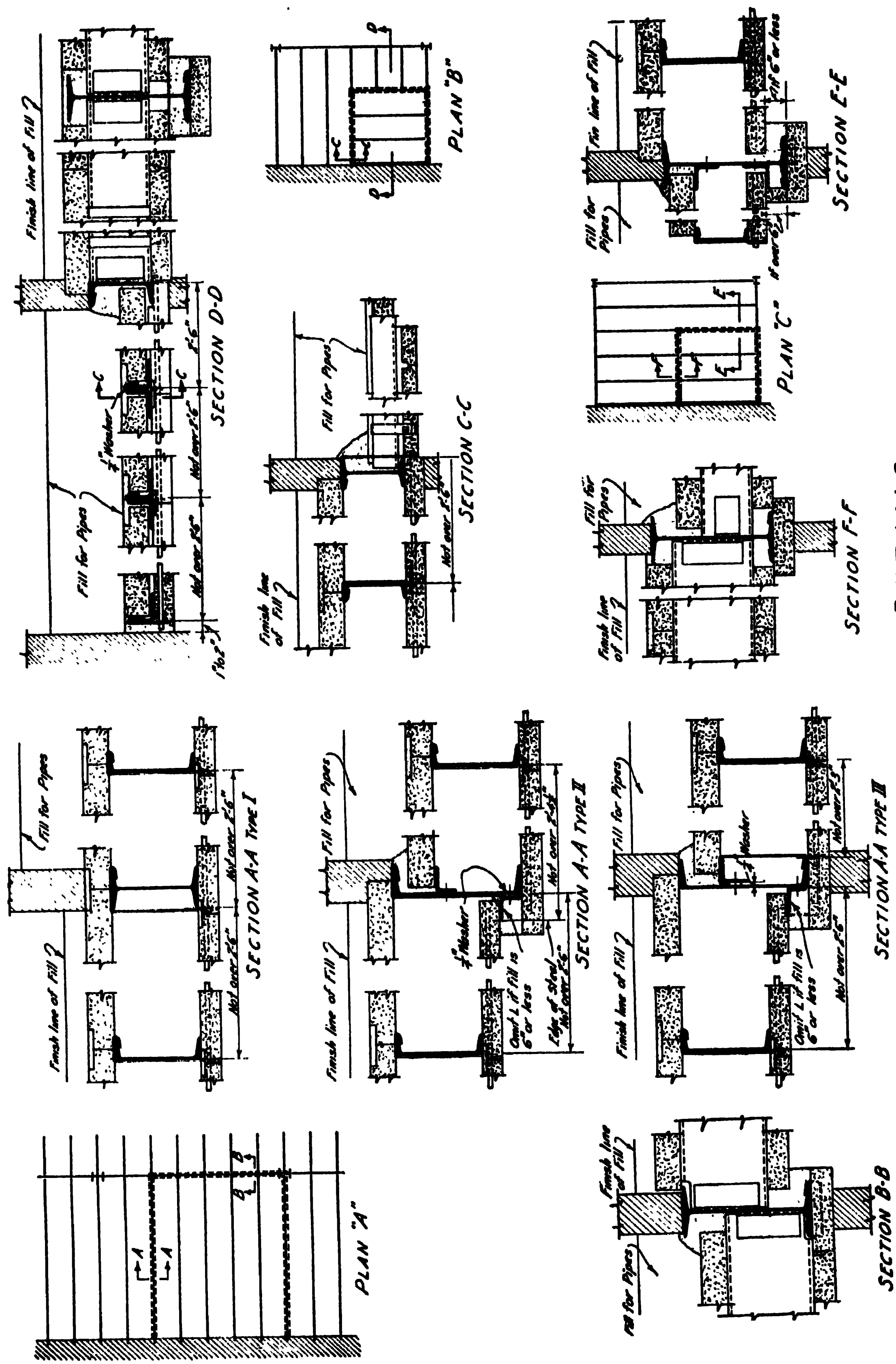

TYPICAL DETAILS FOR PRE-CAST FLOOR CONSTRUCTION
SECTION A-A
When 3s are necessary
SECTION B-B
Finished Floor
Cinder Fill
SECTION D-D
(Through Girder)
TYPICAL FLOOR PLAN
SECTION I-I (See Section C-C)
DETAIL SECTION C-C
Through Channel Joist
SECTION G-G
(at Elevator)
SECTION E-E
Alternate SECTION E-E
SECTION F-F
Alt. SECTION F-F
SECTION H-H
SECTION J-J
SECTION K-K
Alt. SECTION K-K
SPANDREL SECTIONS
Showing method of Fireproofing over Windows
SECTION L-L
SECTION M-M

Fig. 237



TOILET ROOM DETAILS

Fig. 238

Prob. 149a. Determine the sizes of channels, tie-beams, and girders for a typical panel of pre-cast gypsum slab floor construction 20′-0″ square, with a double wood finish floor. Assume 10″ H columns. Make an engineer's sketch of the panel.

150. Poured-in-place Gypsum Slabs.

A variation of the use of gypsum slabs is that of employing the composition discussed in Art. 149, and casting the mixture in forms similar to those for concrete.* The reinforcement consists of two-strand twisted cables of #12 cold-drawn steel wire, spaced from 1″ to 3″ o.c., depending upon the loads and spans. These are anchored at the ends by special devices, as illustrated in Fig. 240. At the center of the span, continuous deflection rods are used normal to the wires, placed 1″ above the bottom of the slab. This feature immediately introduces the principle of the suspension bridge, and **the strength of the slab is dependent upon the tensile strength of the cables acting in suspension.** This construction is similar to that often employed for roofs (Art. 167). Safe loads for slabs under various conditions are given in Table 68.

The construction is probably not as much used as the pre-cast slab type, although it is satisfactory when paneled ceilings are not objectionable. It requires forms, whereas the pre-cast slabs do not, although quick removal of forms is possible. The pre-cast slabs offer the advantage of a flush ceiling. A suspended ceiling could be used for the cast-in-place construction, although it would be more difficult to obtain satisfactory results and would be comparatively expensive. Practically the same advantages may be claimed, otherwise, as have been enumerated for the pre-cast construction.

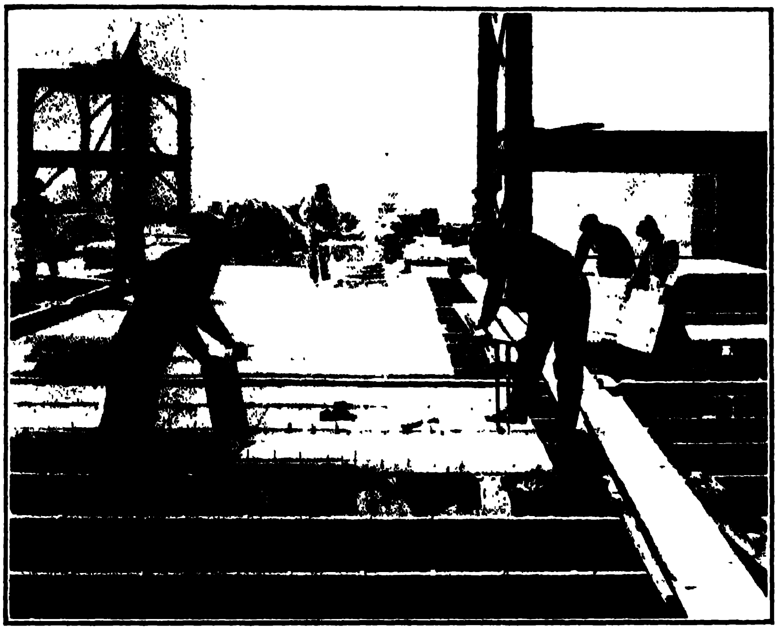


Fig. 239

Certain requirements are necessary for the structural steel frame. The beams should preferably run in a direction transverse to the longer dimension of the building. This gives a better continuity of cable wires. The tops of the beams and girders should lie practically in the same plane. That is,

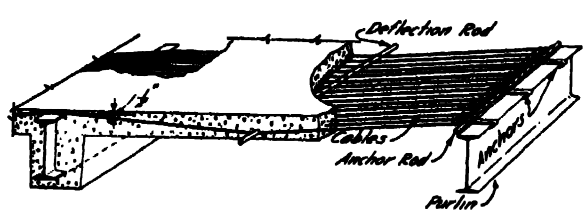


Fig. 240

the tops of the beams may be placed just under the tops of the girders to avoid coping of the former. The girder tops are then protected by the cinder concrete fill only. If more protection is desired, it is necessary to frame the beams and girders flush top and pay for the extra coping. It is necessary to

* These slabs are cast by specially trained workmen of the Structural Gypsum Corporation.

provide channels at the ends of the building to provide anchorage for the cables in the end bays. The pull induced in them by the cables is offset by trussing, as shown in Fig. 241. The toe of the channel should be faced toward the wall and kept 1″ away from it in order to allow anchorage. In the same sense, all openings where the dimension across the cables exceeds 24″ must be surrounded by beams. Openings smaller than this may be taken care of by the use of special rods.

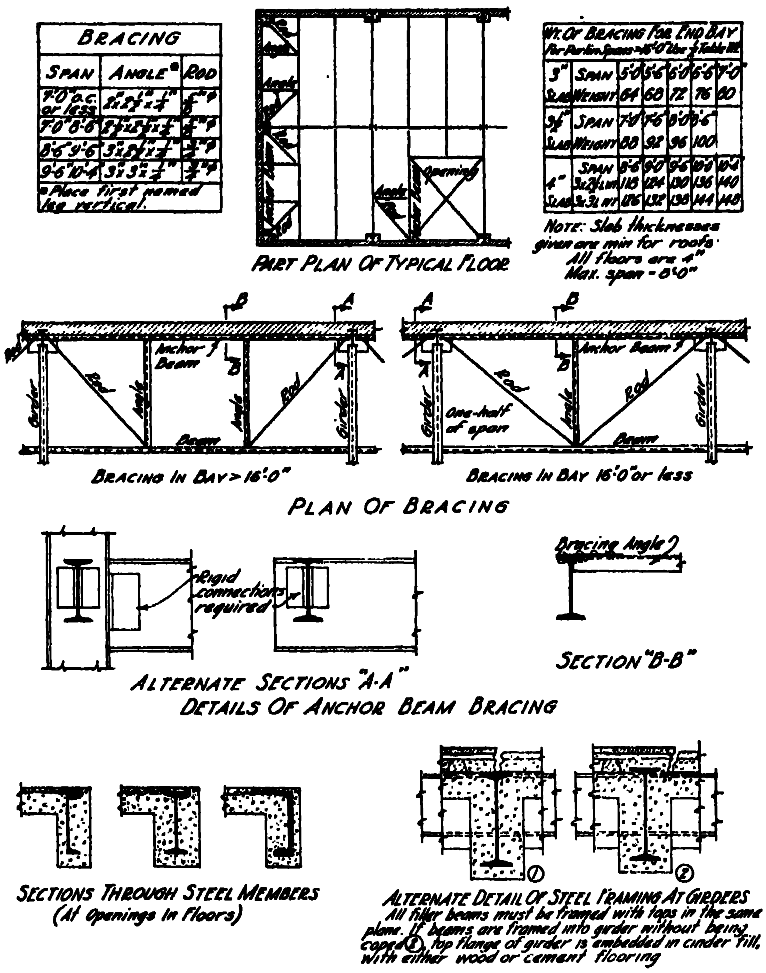


Fig. 241

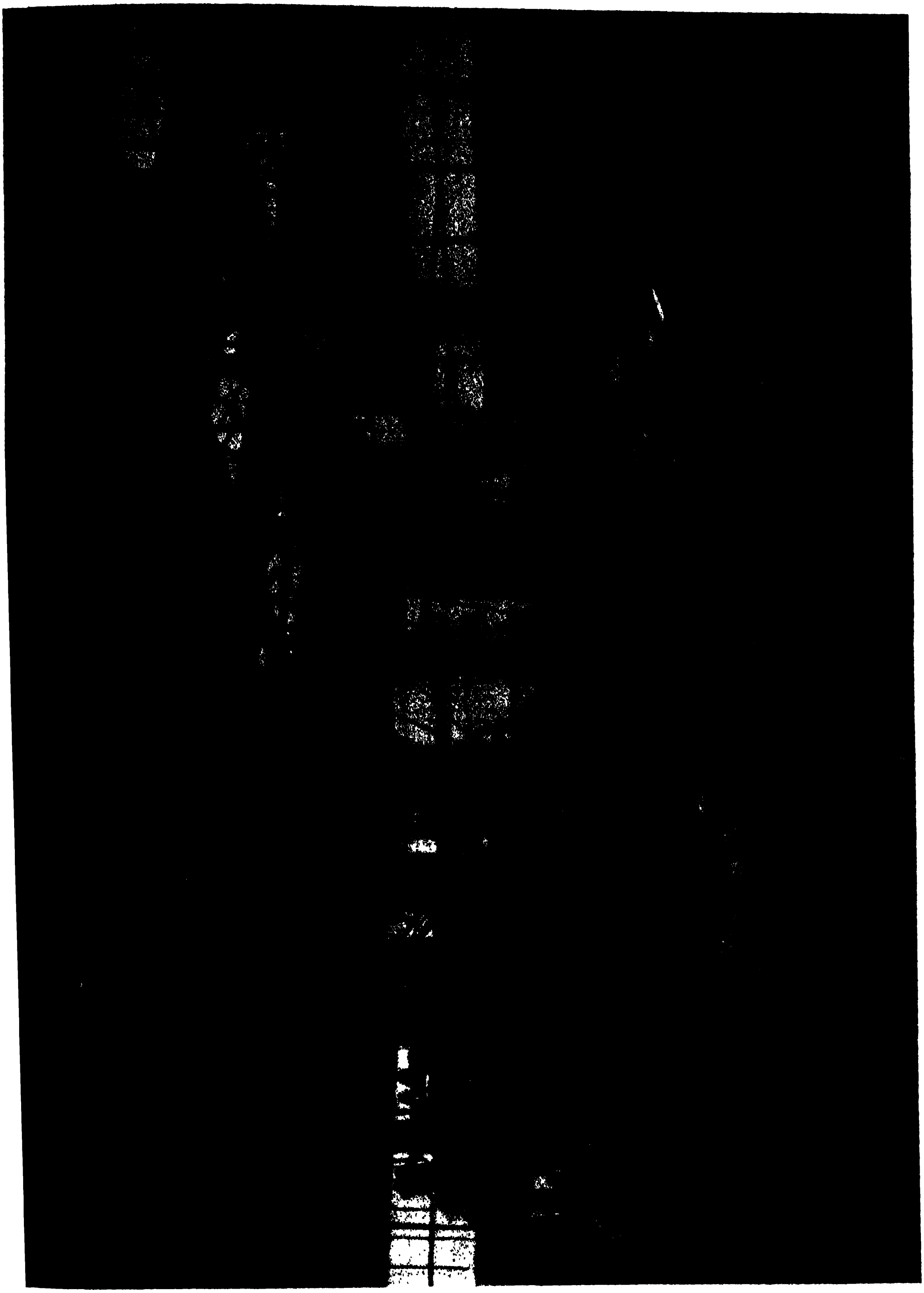


PLATE 23 STRUCTURAL SHOP
NEW ENGLAND STRUCTURAL COMPANY
EVERETT, MASSACHUSETTS

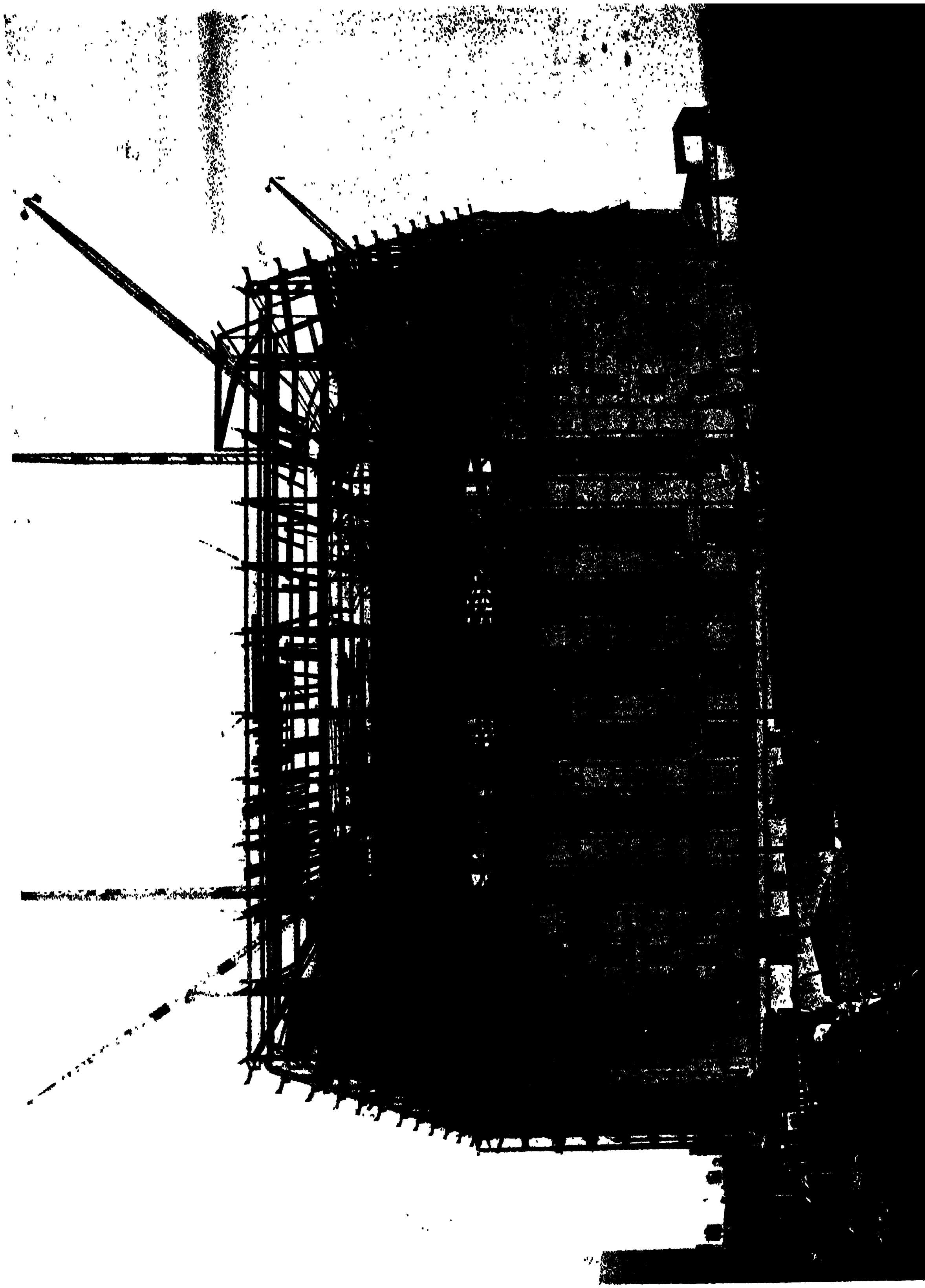


PLATE 24. NEW YORK COUNTY COURT HOUSE, DURING ERECTION

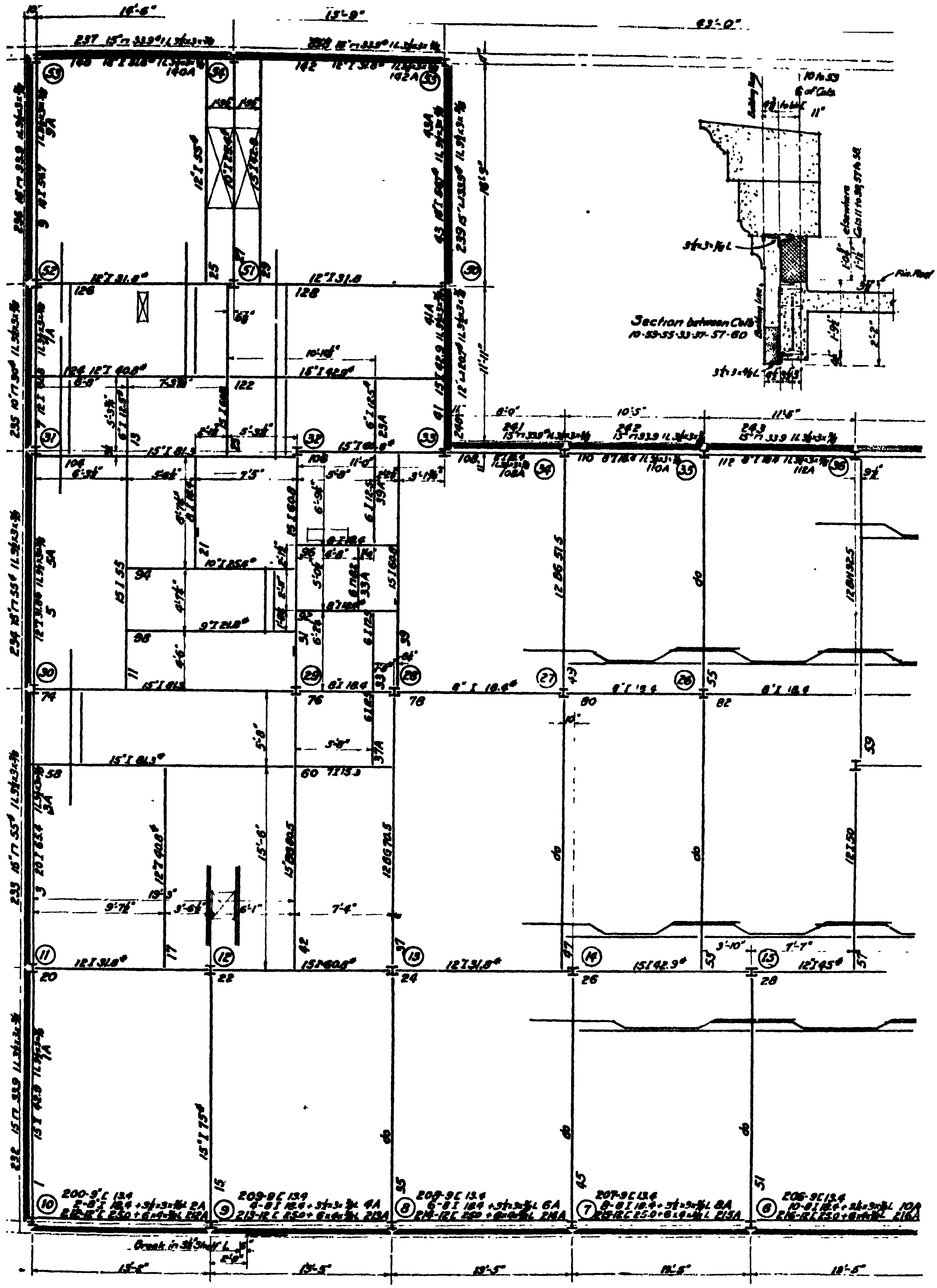


PLATE 25 A TYPICAL ERECTION DIAGRAM

CHAPTER 11

BEAM AND GIRDER DETAILS

151. Erection Diagrams.

After the framing plans have been prepared by the engineer, there still remains a number of definite steps to be made before the details are sufficiently complete to insure the fabrication of the steel members and their erection in accordance with standard practice. The first step which the contractor, to whom the steel work has been awarded, makes, is to prepare erection diagrams of his own, based upon the information given on the engineer's plans (Pl. 21).* A structural engineer must be familiar with such diagrams as he frequently is called upon to approve them before the details are made, or at least before they are completed. Plate 23 shows a view of a typical fabricating shop (see also Pls. 3, 5 and 6). Plate 24 illustrates a job being erected and Pl. 25 shows a typical erection diagram.

On such diagrams, the steel fabricator shows only the steel members and their location. The architect or engineer must approve these locations and verify that they correspond with the intention on the other plans. The column centers are located and assigned numbers, and only a convention is used for the column itself on the plan, — a column schedule (see Index) giving the story heights, sizes of material and splice locations. Each beam and girder is assigned a letter, followed by a number. The letter designates its floor location and the number identifies it in contrast with the others. If two beam details are exactly alike they may be assigned the same number. The letter and the number are called the **beam mark** and this is painted upon the fabricated member to aid the erector. The mark is always placed upon the left hand end of the actual member, so that, on the plan, the indication must be placed accordingly. That is, a beam which reverses from one already labeled would have its mark indicated on the right hand end on the plan. If the fabricator wishes to **substitute a size** from his stock for a given beam as indicated on the engineer's plan, the erection diagram will indicate the proposed change. This must be approved by the architect or engineer. The beam substituted must have an equivalent section modulus and be strong enough to resist the other stresses caused by the load, such as buckling and shear, and must be safe in deflection.

In addition to the beam locations and their numbers, an erection diagram should give the location and the size of wall plates. The latter is often done by assigning numbers to the plates, such as WP1, and so on, and giving a **wall plate schedule** (Art. 15). A note is also given relative to the anchorage of the wall beams (Art. 16). The lintels over the window heads immediately below the floor are also shown in plan and assigned marks such as L1, LL6, and so on. Their locations with respect to the cross-section of the wall are generally shown by sections. In fact, all of the members which are to be furnished by the steel contractor, and the necessary information and dimensions to erect the fabricated parts, must be indicated on an erection diagram for each floor. This includes secondary framing such as tie-rods and permanent bracing. The secondary framing for stairways, elevators, fire-escapes, grilles, railings, trench covers, and the like, is often let in a separate contract and their indications are not commonly a part of the general erection diagrams. Many structural companies have a separate department for this work and class such members as **ornamental work.**†

152. Beam Connections.

There are many varying forms of details used when one steel beam frames into another because of the difference in local practice as to a particular situation, and also because of the many conditions encountered in steel framing. A structural engineer should be familiar with such details in order to avoid awkward framing when possible, and he should be able to discriminate between good and bad connections, as he often has to pass on the details submitted by the steel fabricator for approval.

The most common instance of such work is when two beams of equal size frame into a girder exactly opposite each other in plan and elevation. In the majority of such cases, standard beam connections

* The framing plans made by the engineer must be clear in all respects as to the intent, so that the structural steel fabricator can proceed with his erection diagrams. Framing plans should show the elevations of all floor beams and girders, elevations and locations of spandrel beams, grades of sidewalk framing, dimensions for elevator, stair, vent, flue, and other openings, roof lines, holes for other trades, and so on.

† Additional information is given in other sections relative to the details for stairways, elevators, and so on (see Index).

are used (Art. 28), as illustrated in Fig. 242 (*a*), (*b*) and (*c*). If possible, the beams are placed 2″ below the top of the girder, as in (*a*), to avoid the coping. When beams of different sizes occur on opposite sides, it may be possible that the corresponding standard connection angles will match,

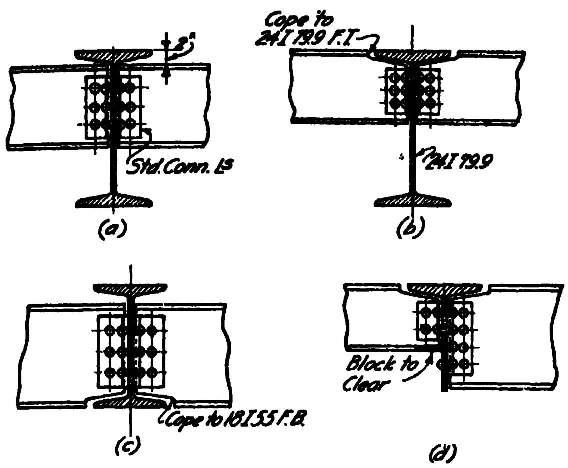


Fig. 242

as shown in (*d*), or special angles may be necessary. When the beams occur at different levels special details such as shown in Fig. 243 may be used. That in (*e*) is preferable to (*d*). When a beam rests

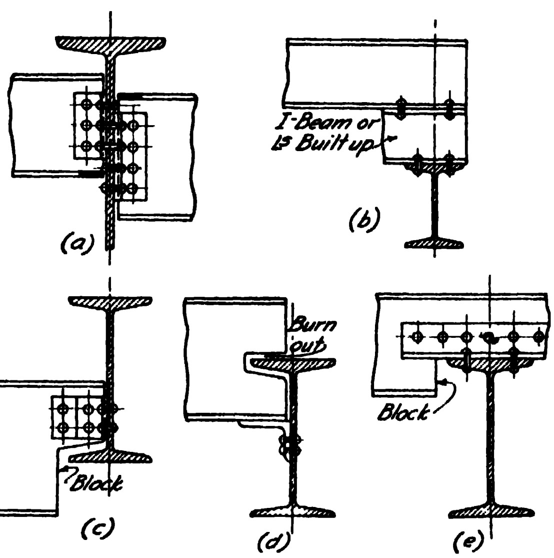


Fig. 243

on top of another the detail in Fig. 244 (*a*) is commonly used, although clips as in (*b*) and (*c*) may be employed. The latter are cheaper but do not provide as much lateral support and are best used to fasten tees and angles only. In Fig. 244, the top beam must be held in position by the construction. Figure 245 illustrates several methods of suspending beams which are below the supporting beam. The nuts is the most common method, as in (*b*), or a bar, as in (*i*). The others are details which are not to be recommended, particularly that in (*g*) as the plate tends to straighten out when subjected to load, and that in (*h*), as this detail is expensive. Figure 246 use of a pair of channels and a rod with washers and

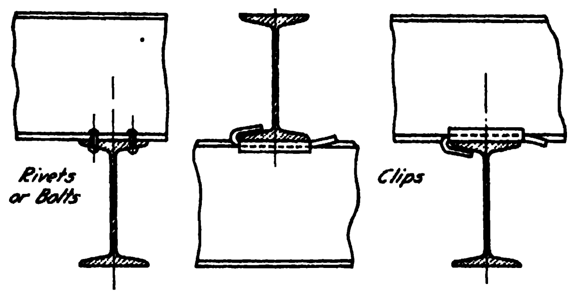


Fig. 244

shows methods of tying a short cantilever into its support and anchor beam. It must be remembered that the tension is in the top of the cantilever and the compression in the bottom. The detail shown in (*a*) is the most common. The compression is resisted by the seat angles and the rivets to the seat angle. The weakening effect of the flange holes should be considered, as well as the effect of blocking the flanges in Fig. 246 (*b*). In (*c*) the rivets must resist the stress as well as prevent

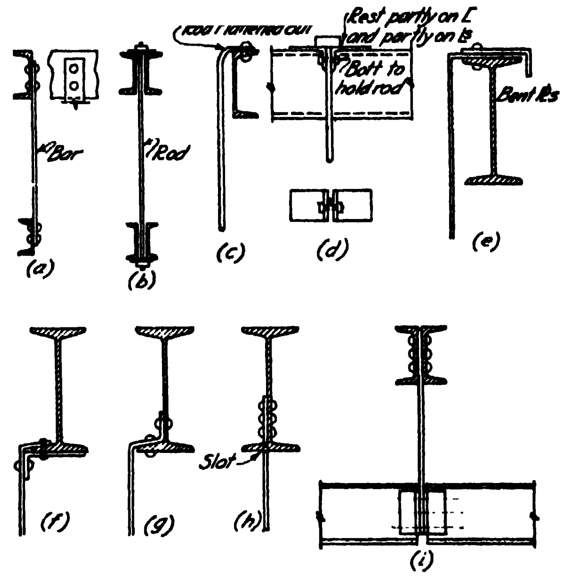


Fig. 245

the plate from buckling. Connections into double beams are shown in Fig. 247. The use of through bolts, as in (*b*), is preferable. Figure 248 gives the limitations for staggered beams. In all connections, eccentricity should be kept to a minimum. Figure 249 illustrates this when a beam frames into a strut. The details in (*a*) and (*c*) are much to be preferred to those in (*b*) and (*d*). The use of a shelf

angle, as in (*e*), does not provide sufficient rigidity to the connection. The details of seat angles and top clips at points where beams or girders frame into columns are discussed in connection with column

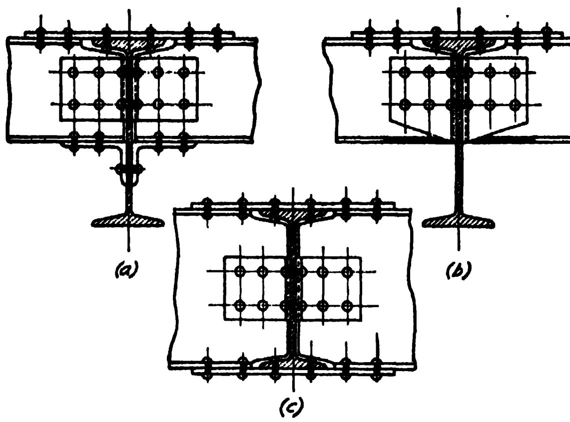


FIG. 246

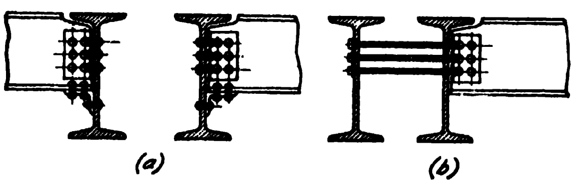


FIG. 247

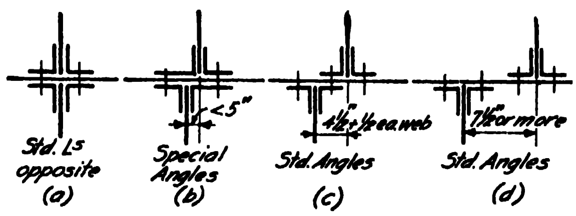


FIG. 248

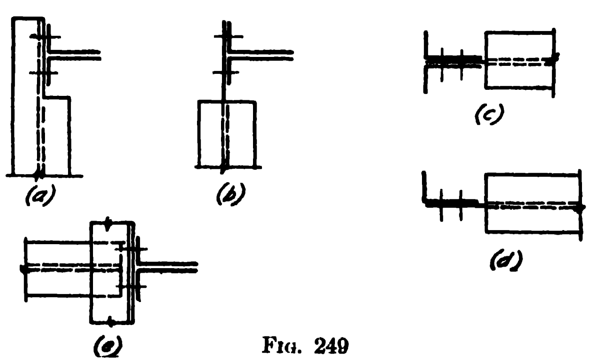


FIG. 249

details, inasmuch as they are a part of the latter details. In all cases, the required number of rivets, their arrangement and their spacing are determined by using the principles outlined in Chap. 3.

Prob. 152a. Design a connection to carry a load of 17,000# if the detail in Fig. 245 (*a*) is used.

153. Shop Drawings.

In order that a steel member may be fabricated properly, it is necessary for the structural steel company to make drawings which will define all the details accurately (Pl. 4). Some of these are the cutting length of the beam, the location and size of the holes to be punched, the size and length of attached connection angles and the necessary cutting away of the beam for clearances. Such drawings are called shop details. The beams are numbered to conform with those on the erection plan, and detailed so that the mark will correspond to the left hand end. Beams which are alike in detail but opposite hand may then be assigned right and left marks such as $B3^R$ and $B3^L$ (Art. 28). The representation of shop details varies with each structural company according to its own particular office standards, but they all embody the same essentials. Plate 4 gives a typical illustration. The figures in the circles at each end designate the distance from the face of the connection angles or the end of the beam to the center line of the member framed into. For a beam framing into a girder, this is one-half of the thickness of the girder web plus $\frac{1}{16}''$. The cutting length of the beam when end connection angles are used is determined by allowing sufficient edge distance for the shop rivets. Standard connection angles and groups of holes to receive other beams are usually represented by standard conventions (Art. 28). All others not standardized must be fully detailed as to location, size and spacing. Figure 250 illustrates typical details.

There are many features in shop details which should be avoided and these must necessarily be learned by experience and actual contact with the work. For large members made up of several pieces, such as plate girders, trusses, and the like, judgment must be exercised as to the number and the location of the field connections. Many members may be shipped as a unit while, in special cases, others are shipped "knocked down."* Decisions will depend upon shipping clearances, the type of conveyance to the job site, the weight of the individual member, the limitations of the usual erection equipment, and the possibility of stresses being developed, in the shipment and the erection, by the weight of the member or its overhanging portions. The dimensioning of holes must be made with regard to a reference plane. When beams frame flush top, the holes should be located with reference to the top flange. In all other cases the holes should be located with respect to the bottom flange as the work

* In export work, special considerations are usually necessary, as ocean freight rates are commonly based upon weight and not by measurement, although when a gross ton of 2240# occupies greater than 40 cu. ft of space, the basis may be that of volume. Additional rates often exist for lengths greater than 30'-0" or weights exceeding 4000#. Projecting pieces are often more susceptible to being bent or broken.

is generally laid out from this plane. In this way, any variation in depth is thrown from the top. Three shop rivets should be used in clip angles when possible, rather than two, to expedite fabrication. removed and the second rivet driven. This insures more accurate location as well as easier fabrication. It should be understood that the above details are given only as typical examples of the many in-

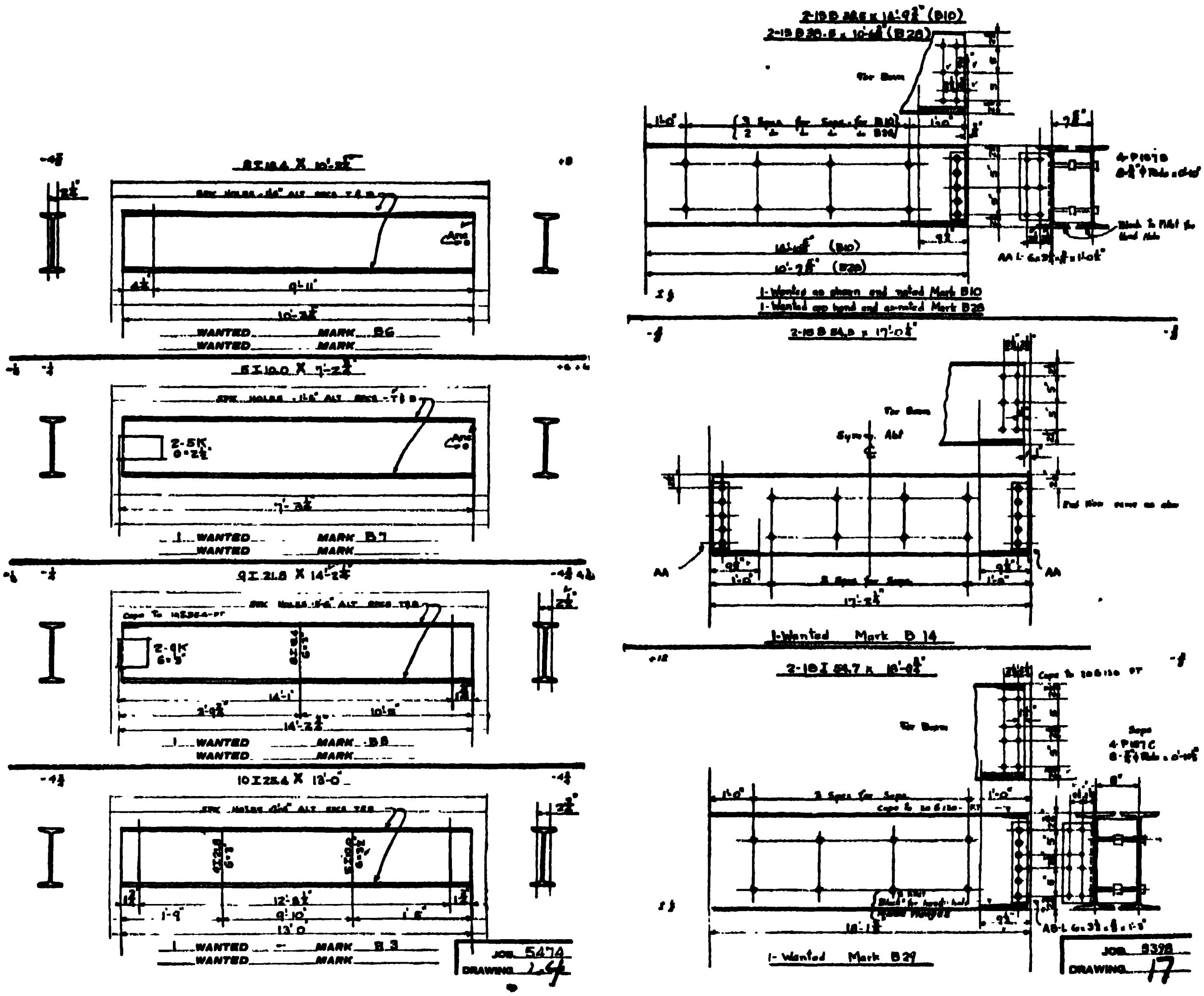


FIG. 250. STRUCTURAL STEEL DETAILS*

(*a*) single beams (*b*) double beams

One hole connections should be avoided, but if they are unavoidable, bolts should be used. When two holes are used, one is temporarily bolted while the other is riveted and then the temporary bolt may be stances when practical rather than theoretical solutions are necessary in shop details.

Prob. 153a. Detail the typical beam and the typical girder in Illustrative Prob. 135*b*.

* Courtesy of Eastern Bridge and Structural Co.

PART III

DESIGN OF ROOF CONSTRUCTION

CHAPTER 12

GENERAL CONSIDERATIONS

154. Requirements for a Roof.

The purpose of a roof is to provide a weather cover which will protect the remainder of a building from the elements. It must be **watertight** in order to resist driving rains and drifting snow and it must have a slope which will provide **drainage** for the water properly. The slopes must be such that pockets, into which snow would drift, are avoided, as the expansion which occurs when water freezes would be likely to cause trouble. The roofing must be held securely to **resist** the tendency of the wind towards **uplift** and of the rain to drive under it. The material of the roof surface should be such that the **heat** action of the sun will **not soften it** materially, as is the case with certain tar preparations. Roof coverings which contain volatile compounds should be avoided, although some protection is often afforded by applying a heat-resisting paint.

The **use of** the **building** has an important bearing upon the type of roof construction. In manufacturing plants in which acid or alkaline processes are common, exposed interior steel will readily corrode and should be eliminated, or otherwise, protected by materials which will resist these gases, or by the use of an anti-condensation lining. The amount of use to which a building is to be put is a factor in determining the materials to be employed for roof work. Whether the building is to be **temporary or permanent,** and the **economic life** of a permanent structure have important relations. In certain instances **portions** of roofs are used **for recreational purposes,** and these require wearing surfaces and heavier construction because of the greater live and incident dead loads.

The **materials** which are used in roof construction must be judiciously selected. The nature of the exposed roofing surface is dependent upon the type of the structure to a large extent. The common kinds of roof coverings are given later in Table 60. Certain ones of these are particularly well adapted to flat roof construction, while others are advisable where pitched roofs are necessary. For instance, shingles or slate are very commonly used for the pitched roofs of residences, and corrugated sheeting is very often employed for certain types of small mill buildings, shops and small warehouses. Similarly, tar and gravel or "ready" roofings are adaptable to flat roofs. The materials used should **conform to** the **architecture** of the remainder of the building, and should also provide ample **fire protection.** The former is determined by the architect in order to produce a harmonious and serviceable combination, such as the use of red Spanish tile with stucco walls, and so on. The fire protection is governed in many instances by building codes, and while some laws allow the use of wood-framed roofs for second- and first-class buildings, certain restrictions are nearly always imposed.

SPECIFICATION CLAUSES*

Section 35. The planking or sheathing of the roof of every building hereafter to be erected or altered shall in no case be extended across the party wall thereof, and every such building and the tops and sides of every dormer window thereon shall be covered with slate, zinc, tin, iron, copper, or such other equally good fireproof material as the Bureau of Building Inspection may authorize; and the outside of every dormer window hereafter placed on any building as aforesaid, shall be made of some fireproof material, and wooden buildings which require roofing, shall not be roofed with any other covering except as aforesaid: Provided, That this shall not apply to roofs and dormers in rural and suburban districts. . . .

Section 9. Semi-detached and detached dwelling houses having sloping roofs of the gambrel or straight pitch types, erected in suburban districts, may have roofs covered with wooden shingles: Provided, No portion of such roofs, including cornice, shall come within eight feet of the party lines: and provided further, That this section shall not apply to mansard roofs.

Section 27. . . . Where the roof is mansard, unless the same is constructed of fireproof material throughout, the lower slope of said (party) wall shall extend at least six inches distant and parallel with the roof covering, and be corbelled to the outer edge of all projections and coped with incombustible material. . . .

In the absence of any definite code restriction, framing materials should be selected which are consistent with the fire risk involved. Thus a

* Excerpted from the "Laws, Ordinances, Rules and Regulations Governing Building Construction in the City of Philadelphia." (Complete to July 1, 1920.)

plank roof carried on steel members, for an iron works, would obviously be hazardous. Outside of such matters of good sense and practice, the framing materials are selected according to comparative prices, availability, ease of placing, and harmony of design.

155. Roof Pitches.

The pitch of a roof may be defined as its rate of slope. This may be expressed in the following ways:

(1) As the **ratio of** the **rise** of the roof **to** its **span,** illustrated in Fig. 251 (*a*). This is a common method and is the usual one referred to directly by the word pitch. A common error is to express a ratio of the rise to the half-span. Correctly stated, a roof is said to be half-pitch, quarter-pitch, full-pitch, and so on, when the rise is $\frac{1}{2}$, $\frac{1}{4}$, 1, etc., times the full width of the building. Thus a quarter-pitch roof with a 40′-0″ span has a 10′-0″ rise.

(2) As the **relation of** the **rise to** the **run,** shown in Fig. 251 (*b*). This is more commonly stated as the rise per foot of run, such as 6″ in 12″.

(3) As an **angle of inclination** in degrees, such as $\alpha = 30°$, shown in Fig. 251 (*c*). This method is very convenient in computations when the angle is one of the commonly met values such as 30°, 45° or 60°.

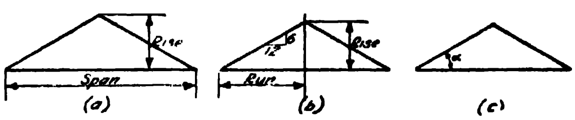


Fig. 251

The desired slope given to a roof is usually determined by the architect, but the engineer must understand the common requirements in order to produce an efficient design. Some of the **influential factors** are the style of the building, the kind of roofing to be used, and the limiting ranges of cost. If the pitch of a roof is definitely established for architectural reasons, there are only certain kinds of roofing material adaptable to it. If a definite roofing material is desired, the pitch of the roof must be limited to a range of slopes which correspond. Roofs may be classed as flat or pitched. Flat roofs are considered as those in which the rise is not greater than $1\frac{1}{2}$″ or 2″ per foot. Table 60 gives the various roofing materials recommended for pitched roofs.

The **cost** of a roof has a definite **relation** to its pitch, and if other conditions will allow, an economical pitch is the object which the designer seeks. In general, the quarter-pitch (6″ per foot) is most economical, as a smaller amount of roofing material is required and a sufficient roof void is available for a truss, if it is structurally necessary. A third-pitch gives a better slope and less effective snow load, but results in a greater wind load and the use of more roofing material. The resultant stresses in a truss covering a third-pitch are larger than in a quarter-pitched roof. In the latter, the lengths of the compression members are comparatively short and thus there is an actual saving of material. For a pitch of $\frac{1}{5}$, the stresses in a truss are greatly increased.

TABLE 60

MINIMUM DESIRABLE PITCHES FOR ROOFING MATERIALS

Roofing	Rise in 12″	Degrees	Pitch	Remarks
Shingles	6	26° 34′	$\frac{1}{4}$	$\frac{1}{4}$ most economical
Slate —				
small	8	33° 40′	$\frac{1}{3}$	$\frac{1}{3}$ preferable (if less, wind raises slate, allowing seepage of rain)
medium	6	26° 34′	$\frac{1}{4}$	
large	4	21° 48′	$\frac{1}{6}$	
Tin —				
flat seams	$\frac{1}{2}$	....	....	Not > 4″ to 12″
stdg. seams	4	21° 48′	$\frac{1}{6}$	$\frac{1}{4}$ preferred
Tile —				Better 10″ to 14″ per foot or flat
plain	7	30° 15′	....	
Spanish, etc.	12	45°	$\frac{1}{2}$	
metal	7	30° 15′	....	12″ better
Corrugated iron	$4\frac{1}{2}$	20° 33′	....	$\frac{1}{4}$ preferable
Vertical crimped steel	2	9° 27′	$\frac{1}{12}$	
Asphalt or asbestos	2	9° 27′	$\frac{1}{12}$	
"Ready" roofing	$\frac{3}{4}$	....	$\frac{1}{16}$	
Copper — stdg. seams	$\frac{1}{2}$	....	$\frac{1}{24}$	
Canvas	$\frac{1}{2}$	....	$\frac{1}{24}$	
Tar and gravel*	$\frac{3}{8}$	....	$\frac{1}{32}$	Maximum not to exceed $\frac{1}{4}$ pitch, as tar runs on being applied or in hot sun, and gravel rolls, leaving exposed spots. Preferable ` pitch — minimum

Another factor, which influences the rise and consequently the pitch, of trussed roofs is the **economical depth of** the **truss.** The following ratios of span and depth are average values:

* Certain roofing companies prefer a perfectly flat roof to lay their material on, as uneven slopes often result if an attempt is made to make the supporting frame conform to a slight bevel. The pitch is made up in the roofing in such cases.

Spans up to 36′, height not less than $\frac{L}{9}$,

Spans 40′ to 80′, height not less than $\frac{L}{10}$,

Spans in general, heights of $\frac{L}{7}$ to $\frac{L}{6}$ are most economical.

If the heights are made less than these ratios, the stresses increase greatly, for a truss is only a form of beam and therefore the deeper the truss, the less the resulting stresses.

156. Selection of the Type of Roof Frame.

The general considerations when the design of a roof frame is undertaken may be classified as follows:

(1) the **span** between the supports which will be consistent with the building as planned,
(2) the **inclination** of the roof surface which will conform to good practice and give the desired appearance,
(3) the **space** to be provided directly **under the roof** and how it is to be finished,
(4) the **basic materials** which will be consistent with the building in general.

Consequently, subdivisions are necessary in the discussion if the problems are to be attacked by a method common with that in actual practice.

When the span, center to center of outside walls or of columns, is greater than 40′-0″ and interior supports are not desired, it usually becomes necessary to plan some other type of frame than that provided by simple beams, to support the roof. Furthermore, the ceiling in the top story, if a necessary architectural requirement, must be supported. When roofs are inclined, " built-up " members* are expensive, as it is necessary to have them conform to the pitch unless an extended form of roof support is used. Where economy and rigidity require large depths, some form of truss is generally employed. This introduces a system of " web members " which affords economy by saving weight, because of the increased depth. The first decision is then whether trusses are necessary or not. From this viewpoint, roofs may be classified for convenience of discussion into:

(1) simple frames, and
(2) complex frames.

Those under (1) may be thought of as beam carrying members with some form of enclosure material and may be either flat or pitched. Those under (2) also involve beam carrying members, but these elements eventually transfer the loads to a truss of some form or other, and here again the roof may be either flat or pitched. The usual procedure in either type of roof (simple or complex) is to design the elements which carry the roofing first. Such computations involve only the principles of simple beams as already discussed (Part I), with special considerations common to roof design. These are considered in Chap. 13, since the methods of design of such members are common to both types of roofs. Chapter 14 refers to simple roof frames in which no trusses are involved, and is divided into two natural sections, namely, pitched roofs and flat roofs. When the type of a simple roof is known, the next question which arises is that of materials.†

The design of the trusses in a complex roof is generally made a separate step in the work. The principles of mechanics are common to all trusses. Figure 252 gives a general range of truss spans and types which will be discussed later. The first decision which must be made is usually that of the kind of material for the truss. This decision is dependent upon pitch and loadings.

157. Kinds of Loads.

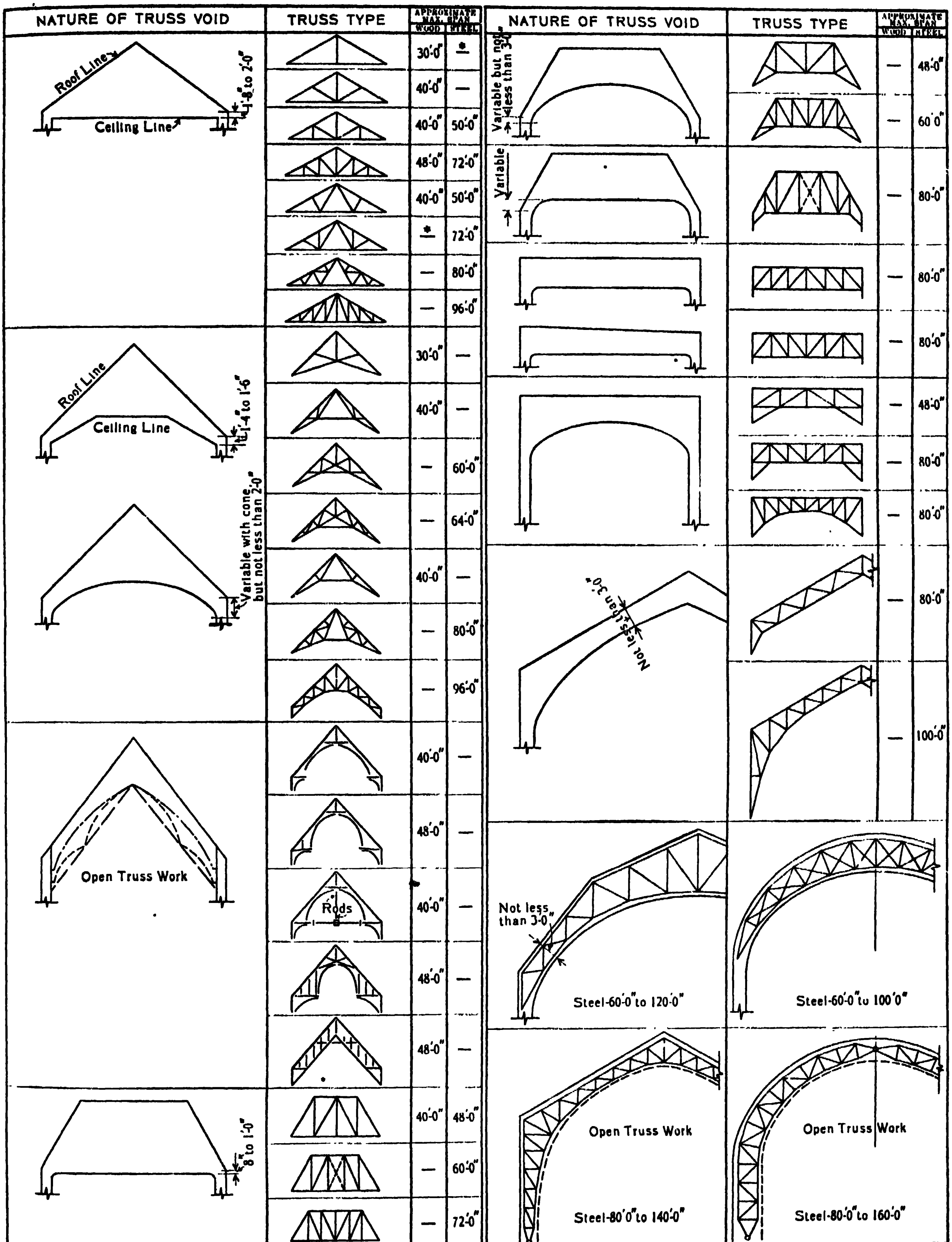
As in all cases of design, there are two general kinds of loads to be considered in the planning of a roof frame, namely, the dead loads and the live loads. The former include all the fixed loads that are either supported by, or suspended from, a roof (Art. 88). These are practically always considered as acting vertically, and in general, represent the weight of the roof structure. Live loads may be classed as snow loads, wind loads, and other temporarily imposed weights. The last include those due to mechanical equipment, such as shafting, hangers, elevators, and so on. In special cases of live loading the usual allowances for snow and wind loads will provide against those occasional loads caused by persons, repair work and the like. Other cases arise when the roof or portions of the roof are to be used for recreational purposes, or when future vertical extension is anticipated. In such cases future floor loads are involved.

158. Dead Loads.

In order to determine the dead loads which must be carried by a roof system, it is necessary to estimate the weights of the materials forming the roof covering, those directly supporting the roofing materials, the roof frame itself, and the trusses, if any are used. These are usually expressed as pounds per square foot (#/□′). In many cases it is also

* " Built-up " is here used to mean such members as compound timber girders, plate girders, and the like.

† The articles in this chapter attempt to differentiate in this respect.



* Where either wood or steel is not customary the span has been left blank. Diagram for wood is opposite to that for steel.

FIG. 252

necessary to include the weight of an attached ceiling (Table 30). The following table gives values for the weights of roof coverings and such framing and enclosure materials upon which the roofing materials rest.

TABLE 61

WEIGHTS OF ROOFING MATERIALS

ROOF COVERINGS	#/□'
Shingles, wood — common	2½
wood — 18″	3
asbestos	2½ to 4½
Slate — $\frac{3}{16}$″ (3″ double lap)	8
¼″ (3″ double lap)	10
Tar and gravel or felt and gravel — 4 ply	8
Tar and gravel or felt and gravel — 5 ply	10
Slag	4
Ready Roofings (Elaterite, Ruberoid, Felt and Asphalt, etc.)	1
Thatched	6½
Copper sheeting	1½
Tin	1
Sheet lead — ⅛″	7
Corrugated iron or galvanized iron	2½
Glass, ¼″ plain or wire	3½
¼″ corrugated	4½
$\frac{5}{16}$″ plain or wire	5
⅜″ plain or wire	6
Clay tile, Spanish, Roman, Ludowici	8
Plain	8 to 10
Oriental	11
Interlocking	14
Non-condensing base	1
Skylight frames	4
Cinder concrete fill for crickets (per inch of thickness)	8
ROOF ENCLOSURE MATERIALS	
1″ Sheathing, hemlock, spruce and the like	3
yellow pine	4
battened	2½
Plank, per inch of thickness (yellow pine)	4
Concrete slab (stone), per inch of thickness	10½
" " (cinder) " " " "	9
Gypsum " " " " "	8
SUPPORTING FRAMES (Average)	
Steel rafters	3
Steel purlins	3
Steel trusses, span 50′-0″ average	4 to 6

159. Snow Loads.

The proper allowance for the loads on a roof caused by snow depends upon the geographical location of the building site, as well as upon the altitude, humidity and climate of that region. That is, the allowance would not be a maximum necessarily where the snowfall is greatest, as a light snowfall, later turned to sleet, weighs more than an increased depth of snow. There are also other factors which are important. The inclination of a roof and the relative roughness of the roofing surface are very influential. The steeper a roof, the greater would be the tendency of the snow to slide off from it, depending, of course, upon whether the character of the roof surface retarded it or not. In connection with the latter feature, a structural designer should determine whether the architect plans to use snow guards or not.*

Dry, freshly fallen snow weighs from 5# to 8#/c.f., and packed snow approximates 12#/c.f. Sleet may weigh as much as 30#/c.f. Light snow may accumulate on flat roofs, in some localities, to a depth of 3′-0″, while sleet probably would never be thicker than 12″ as a maximum. On the basis of the above figures, the maximum loads per square foot for flat surfaces are:

3′-0″ × 8# = 24#/□′ for light snow, and
1′-0″ × 30# = 30#/□′ for sleet.†

The maximum condition might reasonably be set at 30#/□′, but this maximum should not be used for all localities, as the latitude of the place is important. Prof. Ricker gave an empirical formula for this determination as follows:

$$w = 2.5\ (L^\circ - 35^\circ), \text{ in which} \qquad (S\text{–}41)$$

w = the maximum weight for the snow allowance for flat roofs at a given place, and
L° = the north latitude of the locality.

Thus for a given place, such as northern Maine, north latitude 45°, $w = 2.5\ (45 - 35) = 25$#/□′.‡

The preceding values are all referred to flat surfaces, but as stated above, the steeper a roof, the less the active snow load. The following is an example of how the loads may be reduced:

SPECIFICATION CLAUSE §

All slopes up to 20° shall be designed for 25#/□′ of horizontal projection of roof, and this value may be reduced 1# for each degree increase in inclination up to 45°, above which value no snow load need be considered.

Table 62 gives values which are based upon latitudes in general, referring to divisions of the United States, and to various roof pitches. In all cases the snow loads are **considered as acting vertically** and the values refer to the area of the roof surface. Snow loads should be considered as live loads and should not be included with the dead load in the majority of instances, and the extreme conditions should be investigated, as such loads may

* The use of snow guards is discouraged when sufficient property is available to allow the snow to drop upon unfrequented areas. They are very apt to be the cause of leaks in the roof at the eave lines as they tend to concentrate water at these points.

† In tests at Brown University in the winter of 1922–23, a maximum load of 35#/□′ was obtained.

‡ Abstract values are used for L° and the angle of pitch.

§ "The Structural Design of Buildings" by C. C. Schneider, M. Am. Soc. C. E., Transactions, Am. Soc. C. E., Vol. LIV (June, 1905), p. 382.

act only on one side of the roof at any one time. A heavy wind and the action of the sun on the other side may dislodge the snow there, or the snow may be deposited only upon one side by drifting. When the pitch is variable, the snow might stand on part of either or both sides or may often cover the entire roof surface.

TABLE 62*

ALLOWANCE FOR SNOW LOAD #/□′ OF ROOF SURFACE

Location	Pitch of Roof				
	½	⅓	¼	⅕	⅙ or less
Southern States and Pacific Slope..........	0 to 0	0 to 5	0 to 5	5	5
Central States..........	0 to 5	7 to 10	15 to 20	22	30
Rocky Mt. States.......	0 to 10	10 to 15	20 to 25	27	35
Northeastern States....	0 to 10	10 to 15	20 to 25	35	40
Northwestern States....	0 to 12	12 to 18	25 to 30	37	45

Where snow guards are used — allow for ½ pitch, values as given for ⅓ and max. values tabulated.

160. Wind Loads.

The amounts of pressure induced by the action of the wind depend upon a great many variables, such as the geographical location of the building, the height of the surface under consideration above the ground, the velocity and the direction of the wind, protection by other structures and the inclination and the size of the area affected. The first of these, namely the locality, is influential because prevailing winds usually travel in definite paths across the country. Wind velocities do not usually exceed 60 miles per hour (m.p.h.) in the United States except for rare hurricanes and tornadoes. In the latter instances, wind velocities have varied from 80 to 140 m.p.h. For a given average velocity, the velocity at different elevations varies according to the distance, h, above the ground. Stevenson's experiments show that this variation is proportional to $\sqrt{h}$.

The wind is **assumed** to blow in **horizontal** lines, although there is no definite basis for such an assumption. If this were true, the wind would exert its maximum pressure on a vertical surface. Assuming a stream of air of finite section to be impinged upon a flat surface, the area of which is much greater than that of the stream, the following relations may be derived:

Let m = the mass of the air delivered per second on 1□′ of surface,
v = its velocity in feet per second, and
P = the pressure of the air per □′ on a vertical exposed surface.

Then $m \cdot v = P$ = the change of momentum per second, per □′ of air stream.

Let W = the weight of air delivered per □′ per second,
w = the weight of 1 c.f. of air (= 0.0807#), and
g = the acceleration of gravity.

Then $m = \frac{W}{g}$, and

Substituting in $m \cdot v = P$,

$$\frac{W \cdot v}{g} = P.$$

But $W = w \cdot v$. Substituting,

$$P = \frac{w \cdot v^2}{g}.$$

The velocity in feet per second, v, = 1.466 V, the velocity in m.p.h. Substituting,

$$P = \frac{w(1.466\ V)^2}{g} = \frac{0.0807\ (1.466)^2\ V^2}{32.2}.$$

$$P = 0.0054\ V^2. \qquad (1)$$

If, as in the usual case, the section of the wind stream is greater than the surface impinged upon, the change of the direction of the moving air is not complete and consequently the momentum is less. For this reason, the pressure per sq. ft. on a large surface is less than on a small area, and the expression (1) above is excessive. Experiments by Sir Benjamin Baker† showed the ratio of the area to the wind pressure to be about 1.5. On this basis,

$$P = \frac{0.0054\ V^2}{1.5} = 0.0036\ V^2. \qquad (2)$$

No allowance in the above theoretical deduction of (1) was made for the friction of the air on the surface impinged upon. In view of such practical considerations, a great amount of experimentation has been done to check against the theoretical formula. The following are representative:

$P = 0.0032\ V^2$ (From experiments at Eiffel Tower, Paris, and National Physical Laboratories of England).
$P = 0.0040\ V^2$ (Established by Prof. C. T. Marvin from experiments on Mt. Washington).
$P = 0.0036\ V^2$ (Stanton).
$P = 0.0033\ V^2$ (Kernot).
$P = 0.0029\ V^2$ (Dines).
$P = 0.0050\ V^2$ (As given by Smeaton from experiments by Rousse).

The formula which best expresses the average relation between the wind pressure in #/□′ and the velocity of the wind in m.p.h. is

$$P = 0.0032\ V^2. \qquad (S\text{-}42)$$

This represents an average of the above expressions, is based upon a more recent and comprehensive series of experiments, and is adjusted to an average temperature of 60° F. and an atmospheric pressure of 14.7#/□″. For a wind velocity of 100 m.p.h.,

$$P = 0.0032\ (100)^2 = 32\#/□′.$$

Higher intensities than this have been recorded several times,‡ — as much as 60#/□′ and even

* Based upon a table in Mr. F. E. Kidder's "Building Construction," Vol. III., W. T. Comstock Co., Publishers.

† Engineering, Feb. 28, 1890.

‡ Article by Mr. Julius Baier entitled "Wind Pressures in St. Louis Tornado with Special References to the Necessity of Wind Bracing for High Buildings," — Trans. A.S.M.E., Vol. 37, p. 221.

90#/□′ over relatively small areas. However, it would not be expedient to provide against the latter pressures in ordinary structures. It is also unreasonable to expect wind velocities of 100 m.p.h. as usual instances. As in all cases of structural design, values must be decided upon which will provide against unusual conditions, but a limit must necessarily be placed upon such a provision, as the costs would otherwise be unnecessarily high. In view of these considerations, the majority of engineers believe that a maximum pressure of 30#/□′ is sufficient for ordinary conditions, and in many cases this value is reduced.* Some structures are more or less protected by adjacent buildings and a reduction of this value is justifiable for this reason. The possibility of an adjacent structure being removed for one reason or another is always to be considered, but does not seriously affect buildings in city locations.

SPECIFICATION CLAUSE †

All buildings over 150 feet in height and all buildings or parts of buildings in which the height is more than 4 times the minimum horizontal dimension, shall be designed to resist a horizontal wind pressure of 30 pounds for every square foot of exposed surface measured from the ground to the top of the structure, including roof, allowing for wind in any direction.

The above discussion refers entirely to pressures on a vertical plane, whereas in many instances in

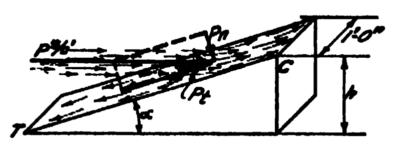


FIG. 253

design, it is desirable to obtain the **effect upon an inclined plane.** On a theoretical basis, the following relation may be established:

For a 1′-0″ length of roof, as in Fig. 253, the horizontal pressure $= P \cdot h \times 1$. The normal component (perpendicular to the slope of the roof) is $P \cdot h \cdot \sin \alpha$. The intensity of this force is

$$P_n = \frac{P \cdot h \cdot \sin \alpha}{TC \times 1}.$$

But $TC = \dfrac{h}{\sin \alpha}$. Substituting,

$$P_n = \frac{P \cdot h \cdot \sin \alpha}{\dfrac{h}{\sin \alpha}} = P \cdot \sin^2 \alpha. \tag{1}$$

This theoretical formula makes no allowance for the reduction of pressure induced on the leeward side, which is known to exist, and the effect of the tangential component, P_t, is neglected. Furthermore no allowance for frictional retardance is made. Consequently, investigators have striven to establish a formula which is more practical and which would agree better with actual conditions. Several formulas have resulted, the most important of which are:

$$P_n = \frac{P \cdot 2 \sin \alpha}{1 + \sin^2 \alpha} \text{ (Col. Duchemin).} \tag{2}$$

$$P_n = P \cdot \sin \alpha^{\,(1.842 \cos \alpha - 1)} \text{ (Hutton).} \tag{3}$$

$$P_n = \tfrac{2}{3}\alpha \text{ (N. C. Ricker).} \tag{4}$$

(To be used for values of α up to 45°, above which $P_n = 30\#/□′$.)

These empirical formulas agree with each other within a few pounds. Ricker's is the simplest, and the Hutton formula the most complex.

Of the above expressions, the Duchemin formula, namely,

$$P_n = \frac{P \cdot 2 \sin \alpha}{1 + \sin^2 \alpha} \tag{S-43}$$

is probably the most frequently used and, it would seem, the most reliable, as it is based upon a very large number of experiments. It gives larger values than some others, but this makes it conservative, although not excessively so. Table 63 gives the

TABLE 63‡

NORMAL WIND PRESSURE, P_n. ($P = 30\#/□′$)

$\alpha°$	$P \cdot \sin^2 \alpha$ (Theoretical) §	$\dfrac{P \cdot 2 \sin \alpha}{1 + \sin^2 \alpha}$ (Duchemin)	$P \cdot \sin \alpha^{(1.84 \cos \alpha - 1)}$ (Hutton)
5	0.0	5.2	3.9
10	0.9	10.1	7.3
15	2.0	14.6	10.5
20	3.5	18.4	13.7
25	5.3	21.5	16.9
30	7.5	24.0	19.9
35	9.9	25.8	22.6
40	12.4	27.3	25.1
45	15.0	28.3	27.0
50	17.6	29.0	28.6
55	20.1	29.4	29.7
60	22.5	....	....
65	24.6	Above 60°	Above 60°
70	26.4	use 30#	use 30#
75	28.0		
80	29.1		
85	29.7		
90	30.0		

tabulated values of P_n for different values of α, based upon the three formulas. From a study of this table the reason why **wind loads are seldom calculated** as acting upon a roof of a **pitch less than 4″ in 12″** ($\alpha = 18°+$), should be obvious. These **values furnish a basis for specifications.** These are generally referred to groups of slopes because small

* C. Shaler Smith in an article in the Trans. A.S.C.E., Vol. 54, p. 37, stated that it is very improbable that a pressure of 30#/□′ extends over a space as wide as 150′ to 200′, and generally a greater pressure than this would not extend wider than 60′.

† The Code of Ordinances of the City of New York.

‡ Based upon Spofford's "Theory of Structures," McGraw-Hill Book Co., Inc.

§ "The values in this column are for purposes of comparison and are not recommended for use for reasons previously stated."

variations in loads are not warranted in such computations. The following is a typical example:

SPECIFICATION CLAUSES*

> Roofs shall be designed to support safely minimum live loads as follows:
>
> Roofs with pitch of four inches or less per foot, a vertical load of forty pounds per square foot of horizontal projection applied either to half or to the whole of the roof.
>
> Roofs with pitch of more than four inches and not more than eight inches per foot, a vertical load of fifteen pounds per square foot of horizontal projection and a wind load of ten pounds per square foot of surface acting at right angles to one slope, these two loads being assumed to act either together or separately.
>
> Roofs with pitch of more than eight inches and not more than twelve inches per foot, a vertical load of ten pounds per square foot of horizontal projection, and a wind load of fifteen pounds per square foot of surface acting at right angles to one slope, these two loads being assumed to act either together or separately.
>
> Roofs with pitch of more than twelve inches per foot, a vertical load of five pounds per square foot of horizontal projection and a wind load of twenty pounds per square foot of surface acting at right angles to one slope, these two loads being assumed to act either together or separately.

In certain instances of design, it is necessary to determine the **horizontal and vertical components** of the normal pressure, P_n. One case of this kind occurs when the end reactions of a truss are wanted. In Fig. 254 (*a*), let P_n = the intensity of the normal

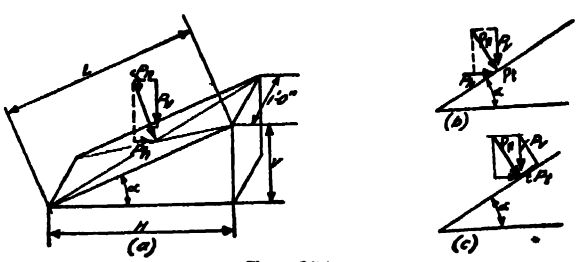


FIG. 254

pressure. The total is then $P_n \cdot L \times 1$. If P_h = the horizontal component of the total normal force and P_v its vertical component,

$$P_h = P_n \cdot L \cdot \sin\alpha \quad \text{and} \quad P_v = P_n \cdot L \cdot \cos\alpha.$$

But $$\sin\alpha = \frac{V}{L} \quad \text{and} \quad \cos\alpha = \frac{H}{L}.$$

Then $$P_h = P_n \cdot L \cdot \frac{V}{L} = P_n \cdot V, \tag{S-44}$$

and $$P_v = P_n \cdot L \cdot \frac{H}{L} = P_n \cdot H. \tag{S-45}$$

The following relations summarize the above formulas:

The horizontal component, P_h, of the total normal pressure is equal to the intensity of the latter, P_n, multiplied by the area of the vertical projection of the roof surface under consideration.

The vertical component, P_v, is, similarly, equal to P_n multiplied by the area of the horizontal projection of the roof surface under consideration.

When the design of certain parts of the roof elements is involved, it is sometimes desirable to obtain the **component parallel to the roof surface.** This may be evaluated in the usual way. Thus, in Fig. 254 (*c*), if P_t is the intensity of the force parallel to the roof,

$$P_t = P_v \cdot \tan\alpha. \tag{S-46}$$

The vertical component, P_v, if resolved, gives the desired force as shown.

161. Combined Loads.

There are times when a combined allowance for the dead weight, snow load and wind load, all applied vertically, may be made instead of obtaining the live and dead load stresses separately and then combining them. Such a method offers the great advantage of saving time, but a designer must know its **limitations** and such a procedure may produce errors which may not be neglected. This method is most frequently used for flat roofs as no wind load need be considered. For the ordinary types of pitched roofs with spans 40′-0″ or less and inclinations not exceeding 45°, the method is sufficiently accurate. Such loads are commonly used in the design of all wood trusses of the V type, for spans of 50′-0″ or less. For most all other cases, particularly for those types of trusses which require counterbracing or which have curved or segmental chords, the roof should be investigated for wind loads and snow loads, separately. In large trusses, the snow and wind load on only one side of the roof may cause reversals of stress which should be provided for.

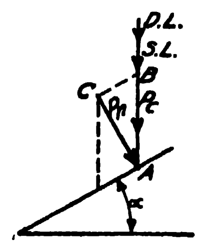


FIG. 255

The use of a combined load virtually assumes that the component P_c in Fig. 255 replaces P_n. The value of the former in the force triangle ABC is larger than P_n. This tends to offset the effect of P_n in the horizontal direction of the roof. To the value of P_c, the value of the snow load (S. L.) and the dead load (D. L.) must be added to obtain the total combined load. It should be understood that this method is fundamentally incorrect, as it is, of course, impossible to have the wind pressure applied on both sides of the roof at once, but the larger vertical

* The Building Law of the City of Boston.

loads require members which are sufficient to carry the stresses resulting from a more careful analysis, for the limiting condition. Many building codes require that no roof or any part of it shall be designed for a total dead and live load of less than 40#/□′, and roofs with a slope less than 25° should be designed for a total load of 50#/□′ as a minimum. Table 64 gives minimum values for combined wind and snow loads. To these must be added the dead load values. This table applies only to the special cases just discussed.

In localities where snow loads are not an important consideration, the following represents good practice:

SPECIFICATION CLAUSE *

Roofs having a rise of 4 inches or less per foot of horizontal projection shall be proportioned for a vertical live load of 30 pounds per square foot of horizontal projection applied to any or all slopes. With a rise of more than 4 inches and not more than 12 inches per foot a vertical live load of 20 pounds on the horizontal projection shall be assumed. If the rise exceeds 12 inches per foot no vertical live load need be assumed, but provision shall be made for a wind force acting normal to the roof surface (on one slope at a time) of 20 pounds per square foot of such surface.

* Report of Building Code Committee "Minimum Live Loads Allowable for Use in Design of Buildings" — U. S. Dept. of Commerce.

Moderate unit loads should be required even where snow is not expected, as roofs having a slope of less than 4″ per foot are always liable to accidental loading, such as groups of people, storage of materials, and so on. Where large snow loads are to be anticipated, the loadings should be increased in accordance with the previous discussion.

TABLE 64†

COMBINED WIND AND SNOW LOADS (#/□′ OF ROOF SURFACE)

Locality \ Pitch	60°	45°	1/3	1/4	1/6	
Northwestern States	30	30	25	30	37	45
Northeastern States	30	30	25	25	35	40
Rocky Mountain States	30	30	25	25	27	35
Central States	30	30	25	25	22	30
Southern and Pacific States	30	30	25	25	22	20

† Excerpted from Mr. F. E. Kidder's "Building Construction," Vol. III, W. T. Comstock Co., Publishers.

CHAPTER 13

ROOF ELEMENTS

162. Definition.

The members of the frame and the enclosure material, exclusive of the truss, will here be designated as elements, and all follow the general theory of beams in their design.

Figure 256 (*a*) gives the names of the component parts of a simple roof, in which no trusses are required. The roofing material is usually supported by sheathing, slabs or units of tile and the like, which, in turn, are carried by rafters. These are

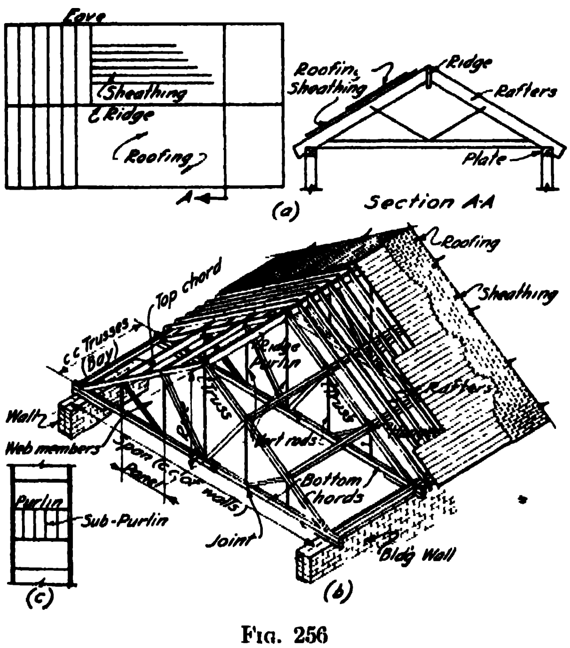


Fig. 256

simple beams placed at a relatively close spacing. They are supported by a member at the ridge and by a wall plate at the eaves. When the span of a pitched roof exceeds 40′-0″, some form of truss, as illustrated in Fig. 256 (*b*), is generally used. The roofing in such cases is carried by planking or slabs which in turn are supported by purlins. In some cases, certain kinds of roofing materials are carried directly by sub-purlins, as illustrated in (*c*). The ends of the purlins are supported by the trusses.

163. Wood Sheathing.

Shingles, slate, tin, tiles, tar and gravel, and other prepared roofings, are usually laid on a wood sheathing. This is seldom greater than 1″ in nominal thickness, or $\frac{13}{16}$″ actual, and is usually from 6″ to 8″ wide. All sheathing is usually of No. 2 common stock.

In cheaper construction, the boarding may be laid open, about 2″ apart, instead of close. This construction involves the use of "batten sheathing," or "roofers." They afford a ready circulation of air around the roof covering but should not be used where the roofing material is flexible. For wood shingles, tile or slate, they effect a considerable saving in lumber. These materials, of course, provide considerable "head cover" and thus offset the tendency toward leakage.

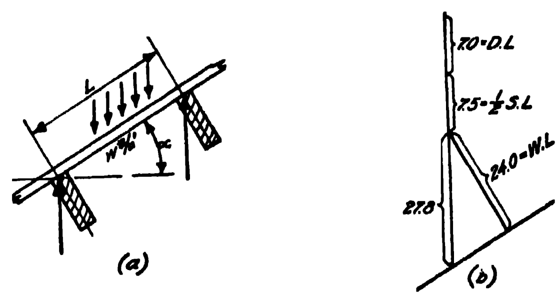


Fig. 257

One of the first steps in the design of a roof is to make sure that the sheathing is safe to span the required distances from rafter to rafter. The fibre stress in bending does not usually define the thickness, but the deflection controls. Tile, slate and similar materials are subject to cracks, and the tongues of the sheathing to breakage, if excessive deflections occur. Protection against occasional concentrated loading, such as people walking upon the roof, must also be provided. Although brittle roofing materials are not as easily affected by deflection as plaster, the safe limit for the deflection is commonly set as $\frac{1}{360}$ of the clear span in inches in all cases.

The sheathing usually extends over several supports, so that limiting conditions may be based upon

beam action which is partially continuous. The moment value for such action is $M = \frac{w \cdot L^2}{10}$. When the load is vertical and the span is inclined, as illustrated in Fig. 257 (*a*), the effective load is $w \cdot \cos \alpha$, and

$$M = \frac{w \cdot L^2}{10} \cdot \cos \alpha \text{ (ft.-lbs.)}$$
$$= 1.2\, w \cdot L^2 \cdot \cos \alpha \text{ (in.-lbs.)}.$$

The more convenient method is to solve for the limiting span as governed by the flexure, L. Thus for 1″ sheathing,

$$M_r = \frac{s_w \cdot b \cdot d^2}{6} = \frac{s_w \times 12 \times (\frac{13}{16})^2}{6} = 1.32 \times s_w \text{ (in.-lbs.)}.$$

As the woods used for sheathing are either spruce or pine, the average value of s_w may be safely taken as 1000#/□″. Hence

$$M_r = 1.32 \times 1000 = 1320''\#.$$

Equating $M_r = M_s$,

$$1 \cdot 2\, w \cdot L^2 \cos \alpha = 1320, \text{ from which}$$

$$L = \sqrt{\frac{1100}{w \cdot \cos \alpha}} \text{ (safe flexure — 1″ sheathing)}. \qquad (S\text{–}47)$$

Similarly, a limiting span for safe deflection may be established as follows:

$$D = \frac{5\, W \cdot l^3}{384\, E \cdot I} \times \frac{8}{10} = \frac{l}{360} = \frac{12\, L}{360}$$

$$\frac{5\,(w \cdot \cos \alpha \cdot L) \cdot L^3 \times (12)^3}{384 \times 1{,}000{,}000 \times \frac{12 \times (\frac{13}{16})^3}{12}} = \frac{L}{30}$$

$$L = \sqrt[3]{\frac{715}{w \cdot \cos \alpha}} \text{ (safe deflection — 1″ sheathing)}. \qquad (S\text{–}48)$$

By comparing the above formulas it is seen that the deflection always controls.

It is usually unnecessary to design sheathing, but when exceptionally heavy loads or wide spacings between the supports are used, it should be investigated. Table 65 gives limiting spans for varying conditions. It may be seen from a study of this table that with ordinary spacings of 16″ to 24″, the ordinary 1″ sheathing is well within the safe limits of flexure.

Illustrative Prob. 163a. Determine whether 1″ nominal sheathing is satisfactory for the following data:

D. L. = 7#/□′. Span 24″. Slope 30°.
S. L. = 15#/□′ of horizontal projection.
W. L. = 24#/□′ of roof surface.

It is necessary to determine the maximum condition of loading. One case is dead load and full snow load, or 7 + 15 = 22#/□′. The other is dead load, ½ snow load and wind load as illustrated in Fig. 257 (*b*). The latter is larger, being 7 + 7.5 + 27.8 = 32.3#/□′.

$$L = \sqrt[3]{\frac{715}{w \cdot \cos \alpha}} = \sqrt[3]{\frac{715}{32.3 \times 0.866}} = 2.92' = 35'' \text{ (allowable)}$$

l = 24″ actual — O.K.

TABLE 65*

LIMITING SPANS IN FEET FOR 1″ SHEATHING FOR VARIOUS LOADS AND SLOPES

s_w = 1000#/□″ E = 1,000,000#/□″ d = 1.0″

Load #/□′	Slope of Roof (in. per ft.)						
	0	2	4	6	8	10	12
20	4.53	4.56	4.60	4.71	4.81	4.95	5.08
25	4.19	4.22	4.25	4.35	4.45	4.58	4.70
30	3.95	3.97	4.00	4.11	4.20	4.32	4.43
40	3.59	3.61	3.64	3.73	3.82	3.92	4.03
50	3.34	3.36	3.40	3.47	3.55	3.65	3.75
60	3.13	3.15	3.17	3.25	3.33	3.42	3.52

Values limited by deflection.
For values of E other than 1,000,000#/□″, multiply tabular values by $\sqrt[3]{\frac{E}{1{,}000{,}000}}$.
For limiting span or sheathing of other than 1″ thickness multiply tabular values by thickness of sheathing in inches.

Figure 258 shows several common details for fastening sheathing to purlins of wood and steel.

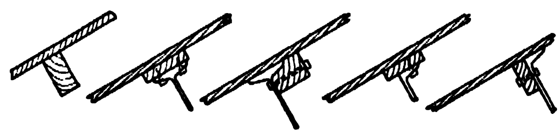


Fig. 258

164. Corrugated Steel Sheets.

For certain types of structures, such as mill buildings, sheds, grain elevators, and small warehouses, corrugated steel sheets make an economical and satisfactory roofing. Figure 259 (*a*) illustrates such material, — the corrugations running lengthwise of the sheets. These are usually laid directly upon steel roof purlins (commonly angles), and held in place by clips of steel hooping, 12″ o.c., wound around the purlins, as illustrated in Fig. 259 (*b*), although clinch nails may be used for angle purlins, as in (*c*).

The sheets are made in 5″, 3″, 2½″, 2″, 1¼″ and ⅝″ nominal corrugation widths, but the 2½″ width (actual 2⅔″) is most generally used for roofing work.† The depth of the corrugation for this variety is ½″, and the width of the full sheet is 27½″. Specifications usually require a side lap of 1½ corrugations, so that the covering width is 24″. The reason for this specification should be obvious from Fig. 259 (*d*). The standard lengths are 5′ to 10′ by 1′-0″ increments, and a lap of 6″ in length is customary for the usual quarter pitched roof. The length most commonly used is 8′-0″. For roofs, the thickness of metal generally used is #20 or #22, U. S. Standard gauge. When wood purlins and close wood sheathing (Book I) are employed, #26 or #28 gauge metal may be used. In this instance 8d barbed nails, 12″ o.c., are used as a fastening. The sheets should be galvanized or well painted with graphite or asphalt paint to protect them against corrosive gases from within and the weather from without.

* From Hool and Johnson's "Handbook of Building Construction," copyright McGraw-Hill Book Co., Inc., New York.

† Data on dimensions, covering widths, areas in square feet of one sheet, number of sheets to cover 100□′, and weights may be found in the "Pocket Companion," Carnegie Steel Co.

SPECIFICATION CLAUSE

The distance between centers of purlins shall not exceed 6′-0″, and preferably shall be made less than this value.

The above limit is based upon the following:

"By experiment it has been determined that corrugated sheet steel, $\frac{5}{8}$″ deep and 0.035 inch thick, spanning 6′-0″, began to give a permanent deflection with a load of 30 pounds per sq. foot, and that it collapsed with a load of 60 pounds per sq. foot."*

Equating,

$$\frac{W\cdot l}{8}=\frac{12{,}000\times 4\,b\cdot d\cdot t}{15},\quad\text{and}$$

$$W=\frac{25{,}600\,b\cdot d\cdot t}{l}.\qquad (S\text{–}49)$$

Another formula which expresses the safe load per square foot, w_0, is

$$w_0=\frac{330\,s\cdot b\cdot t}{l^2}.\qquad (S\text{–}50)$$

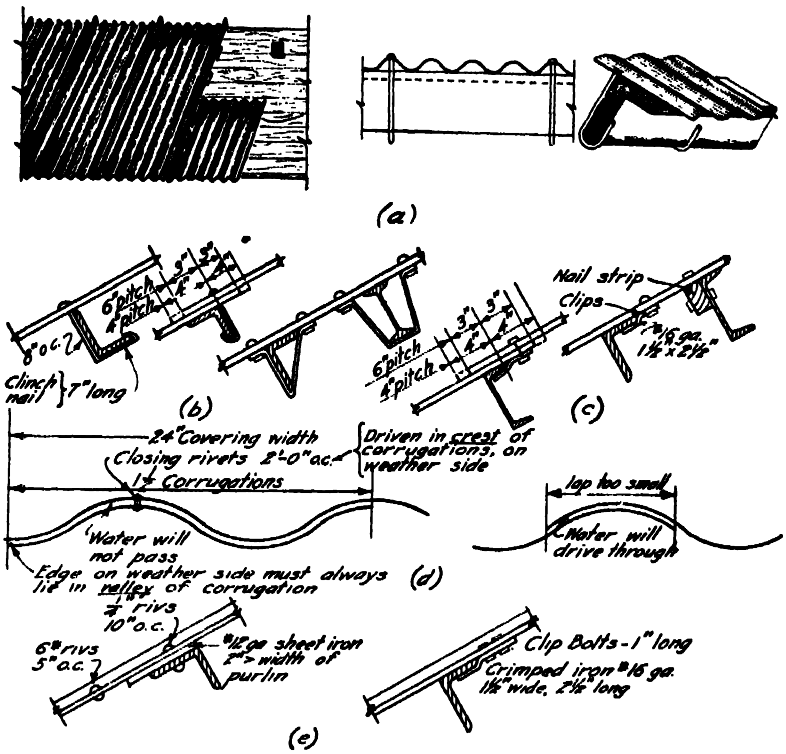


FIG. 259

Some of the common spacings used in practice are as follows:

#24 gauge — 2′-0″ to 2′-6″ c.c. of supports.
#22 and #20 — 2′-0″ to 3′-0″ c.c. of supports.
#18 gauge — 4′-0″ to 5′-0″ c.c. of supports.

An expression* for the approximate total uniformly distributed load in pounds, W, which a corrugated sheet can sustain may be developed in the following manner:

In Fig. 260, let

b = the curvilinear width of the sheet in inches (= 1.075 × the covering width),
l = the unsupported length of the sheet in inches,
t = its thickness in inches,
d = the depth of the corrugations in inches, and
s = the allowable fibre stress, usually taken as 12,000#/□″ (material from puddled iron sheeting and bent hot).

$$M=\frac{W\cdot l}{8}\text{ in.-lbs.}\qquad M=\frac{s\cdot I}{c},$$

$\frac{I}{c}$ (approximately) $\frac{4\,b\cdot d\cdot t}{15}$

* "Pocket Companion," Carnegie Steel Co.

Illustrative Prob. 419a. Determine the safe uniform load in #/□′ that 2½″ standard corrugated sheets, #22 gauge, will sustain, if the clear span is 4′-0″. Use formula S–49. Compare with the result by formula S–50.

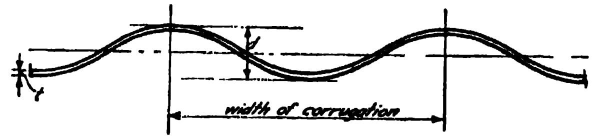


FIG. 260

#20 gauge, $t=\frac{1}{32}$″ $\quad b = 1.075\times 24 \quad d = 2.5$

$l = 48$″

$$W=\frac{25{,}600\times 1.075\times 24\times 2.5}{48}\times\frac{1}{32}=1080\#$$

$$\frac{48\times 24}{144}=8.0\square' \qquad \frac{1080}{8}=135\#/\square'\ \text{(by } S\text{–49).}$$

$$w_0=\frac{330\times 12{,}000\times 1.075\times 24\times 1}{48\times 48\times 32}=138\#/\square'\ \text{by } S\text{-50.}$$

Corrugated sheets generally do not have to be designed for usual loads and ordinary spans, and tables may be used. An expression which was developed by Rankine gives the limiting span in feet for a given corrugated sheet and a definite load in #/□′, w_0, as

$$L = \sqrt{\frac{0.18\, s \cdot b \cdot d \cdot t}{w_0}}. \qquad (S\text{–}51)$$

Table 66 gives several values based upon the above formula.

TABLE 66*

LIMITING SPANS FOR CORRUGATED STEEL SHEETS

$s = 12{,}000$ #/□′ $\quad b = 12''$ $\quad d = \frac{1}{2}''$

Gauge	t (ins.)	Values of L in feet					
		$w_0 = 20$	$w_0 = 25$	$w_0 = 30$	$w_0 = 40$	$w_0 = 50$	$w_0 = 60$
16	$\frac{1}{16}$	7.1	6.3	5.8	5.0	4.5	4.0
18	$\frac{1}{20}$	6.3	5.6	5.2	4.5	4.0	3.6
20	$\frac{3}{80}$	5.5	4.9	4.5	3.9	3.5	3.2
22	$\frac{1}{32}$	5.0	4.5	4.1	3.5	3.2	2.9
24	$\frac{1}{40}$	4.5	4.0	3.7	3.2	2.8	2.6

165. Roof Plank.

In mill buildings and slow-burning construction, plank is often used as a material to support the roofing. In such construction, the rafters are omitted and the planks are spanned between the purlins, or in some cases from truss to truss. The simple method to use in this case is to determine the total effect of the load in a direction perpendicular to the plank and then to design such material in the same manner as for floor construction (Book 1).

Illustrative Prob. 165a. Determine the necessary thickness of yellow pine plank (Grade I) to span 8′-0″. Snow load, 15#/□′ of horizontal projection. Wind load, 24#/□′ of roof surface. Slate roof, 30° slope.

Slate = 7
Roofing paper = 1
2″ Plank = 7
D. L. = 15
S. L. = 15 (full snow load)
T. L. = 30#/□′ vertical

26#/□′ normal component (Fig. 261 (*a*)).

D. L. = 15
½ S. L. = 7.5 (one-half snow load)
Sum = 22.5#/□′ vertical
W. L. = 24.0 (wind load)

43.5#/□′ normal component (Fig. 261 (*b*)).

$$M = 1.2\, w \cdot L^2 = 1.2 \times 43.5 \times (8)^2 = 3340''\#$$

$$3340 = \frac{s_w \cdot b \cdot d^2}{6} = \frac{1500 \times 12 \times t^2}{6} \qquad t = 1.06''$$

$$D = \frac{5\, W \cdot l^3}{384\, E \cdot I} \times \frac{8}{10} = \frac{5\,(43.5 \times 8) \times (8)^3 \times (12)^3 \times 8}{384 \times 1{,}500{,}000 \times \dfrac{12 \times (1.62)^3}{12} \times 10}$$

$D = 0.49''$ actual

$$D = \frac{l}{360} = \frac{8 \times 12}{360} = 0.28'' \text{ allowable}$$

Use 3″ plank on a/c Deflection.

* Based upon a table in Hool and Johnson's "Handbook of Building Construction," copyright, McGraw-Hill Book Co., Inc.

Here again Table 67 may be used to determine the thicknesses. Ordinary matched plank shown in Fig. 261 (*c*) may be employed, or in some instances, shiplap, shown in (*d*), is used. The details at the points of support are similar to those shown in Fig. 258.

TABLE 67

MAXIMUM SPANS FOR ROOF PLANK

Nominal Thickness	Actual Thickness	Fibre Stress #/□″	Span in Feet L.L. in #/□	
			50#	100#
3″	$2\frac{5}{8}''$	1200	13′-8″	10′-1″
"	"	1300	14′-3″	10′-6″
"	"	1500	15′-4″	11′-3″
"	"	1600	15′-10″	11′-8″
"	"	1800	16′-9″	12′-4″
"	"	Deflection	9′-0″	7′-4″
4″	$3\frac{5}{8}''$	1200	18′-5″	13′-8″
"	"	1300	19′-2″	14′-3″
"	"	1500	20′-7″	15′-4″
"	"	1600	21′-3″	15′-10″
"	"	1800	22′-7″	16′-9″
"	"	Deflection	12′-3″	10′-1″

Modulus of elasticity 1,620,000 #/□″.
Deflection limited to $\frac{1}{30}''$ per foot of span.
Matched and dressed plank.
Values based on actual thicknesses.
Sum of L.L. and weight of plank included in calculating the spans.

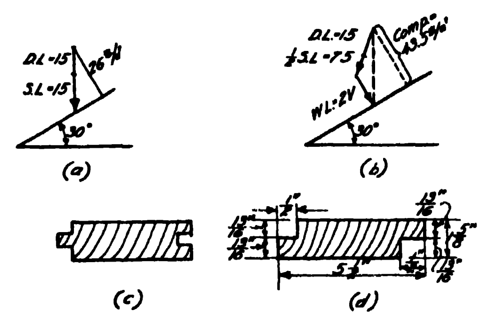


FIG. 261

Prob. 165b. What thickness of yellow pine plank is required to span 6′-0″ and carry a L.L. of 40#/□′ (horizontal) and 1″ roofers, with tar and gravel roofing?

166. Concrete Slabs.

For types of buildings where the fire hazard has an important bearing, concrete slabs may be used to act as a carrying medium between the purlins, instead of the plank or sheathing. The concrete is usually made with a cinder aggregate instead of the stone, for the following reasons:

(1) A lighter dead weight is obtained,
(2) A cinder concrete slab of the usual thickness which conforms with recognized practice is

generally amply sufficient for the loads to be supported,

(3) Roofing materials may often be nailed directly to cinder concrete.

When the loads normal to the slab are correctly determined, the design is similar to that discussed in Chap. 7.

167. Gypsum Slabs.

A material which is finding popular use for roof slabs is gypsum. This may be employed in pre-cast units or it may be cast in place. An important advantage in either instance is that the material is relatively light in weight and sufficiently strong for ordinary roof loads. The pre-cast units are used much more than gypsum cast-in-place on account of the simplicity of the former. Fireproofing of beams is generally omitted in roofs of this kind, especially where trusses are used, as it is not feasible to fire-proof the trusses, and no gain is made unless all steel is protected.

A form of **pre-cast** gypsum unit is the patented Pyrobar* which is made of calcined gypsum and a small percentage of wood fibre. The common size is 3″ × 12″ × 30″ and it weighs about 13#/□′. It is laid directly upon sub-purlins (usually 2¼ × 2¼ × 4.2# T's) 2′-6½″ o.c., and on 5′-0″ spans. The joints at the T's and between the blocks are filled with grout.

If gypsum is to be used and **cast-in-place,** the advantages claimed are that the material sets quickly so that an early removal of forms is possible, and considerable heat is developed during the setting which is an aid to work in cold weather. The principles involved in the design are similar to those of reinforced concrete as discussed in Chap. 7. A common construction consists of a 4″ slab supported by T-joists 6″ wide and 16″ o.c., the depth of the latter depending upon the loads and the spans. The average span used is 10′-0″. In the design, no part of the web of the T-joist is considered as compression area, and steel bars are used in the ordinary way. Wire mesh is commonly placed in the bottom of the slab, principally to offset temperature stresses. The working stresses often employed are as follows:

$f_c = 350\#/\square''$, $f_s = 16{,}000\#/\square''$, $n = 30$,
$u = 30\#/\square''$, $v = 20\#/\square''$, bearing $= 300\#/\square''$.†

It is inadvisable to use web reinforcement, and the web of the T-joists is proportioned to resist the shear without such steel.

Another patented form of similar construction is the **Gypsteel** roof.‡ This is varied by using:

(1) poured-in-place roofs, or
(2) pre-cast slab roofs.

Figures 262 and 263 show typical sections of the first type. Where complete fire protection is required, or where especially severe conditions of humidity and high temperature might cause condensation to form upon the projecting webs and flanges of the purlins, if exposed, the latter may be protected by the gypsum, as illustrated at the left. The following‡ is descriptive of this construction:

"The design of gypsteel construction is based upon the principle of the suspension bridge. Cables of cold-drawn steel wire are spaced from 1″ to 3″ apart (depending upon the spans and loads) and are securely anchored to both ends of a series of purlins by means of bars and anchors of a section of metal sufficiently heavy to develop the strength of the cables. These cables are put into uniform deflection and tension between each pair of purlins by means of continuous steel deflection-rods.

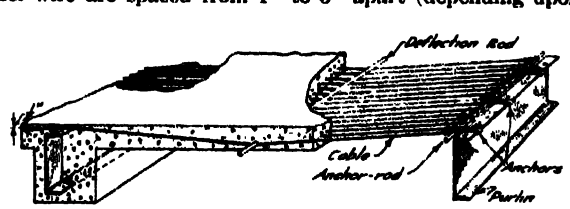


FIG. 262

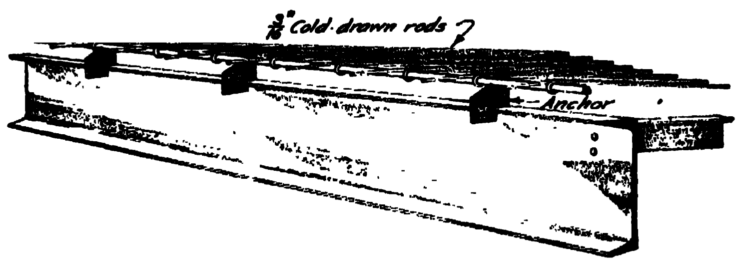


FIG. 263

"Gypsteel composition is then poured in place between the purlins, upon wood forms or centering, and is brought to a level surface about ½″ above the top flanges of the purlins. It is then ready to receive the waterproof roofing.

"This composition consists of a very high grade, scientifically calcined gypsum, with which is incorporated wood fibre or chips that serve as a binder and impart to the slab its peculiar toughness and elasticity."

Figure 264 illustrates the pre-cast slab roofs. The following‡ is descriptive of this type:

"The gypsteel slabs are moulded at the factory in steel forms, into which the steel cables are placed and put into uniform deflection and tension by means of the deflection-rods, restrained at the sides of the moulds. These cables consist of 3-16″ cold-drawn wire rods, spaced from 2″ to 4½″ apart, according to the spans and the loads to be carried. The cables are securely anchored to the moulds at points beyond the ends of the slab, being removed from this anchorage when the gypsum has set, each cable emerging from the ends of the slab within ¾″ of the top surface and projecting about 2½″.

"When these slabs are set in place upon the purlins, with their ends abutting, the projecting ends of the opposite cables in each slab are tied together by a mechanical device which draws them up taut at these connections; and as the ends of the slab are rabbetted where the cables emerge, these ties lie in a depression which is filled with a grout of the same

* U. S. Gypsum Co.
† For nomenclature, refer to Chap. 7.
‡ Structural Gypsum Corporation, New York City (formerly the Gypsum Department of the H. H. Robertson Co.).

gypsum composition, firmly embedding the ties, and troweled off flush with the top surface of the slab.

"Obviously, the resulting roof is, in effect, a monolith, with continuous, securely-anchored supporting cables in deflection and tension, carrying the loads, exactly as in the gypsteel poured-in-place roof."

degree, and is fairly elastic. The gypsum offers good protection to the steel against corrosion. Note the cable ties and the grouting in the side as well as the end joints.

The strength of such roofs can be proven by

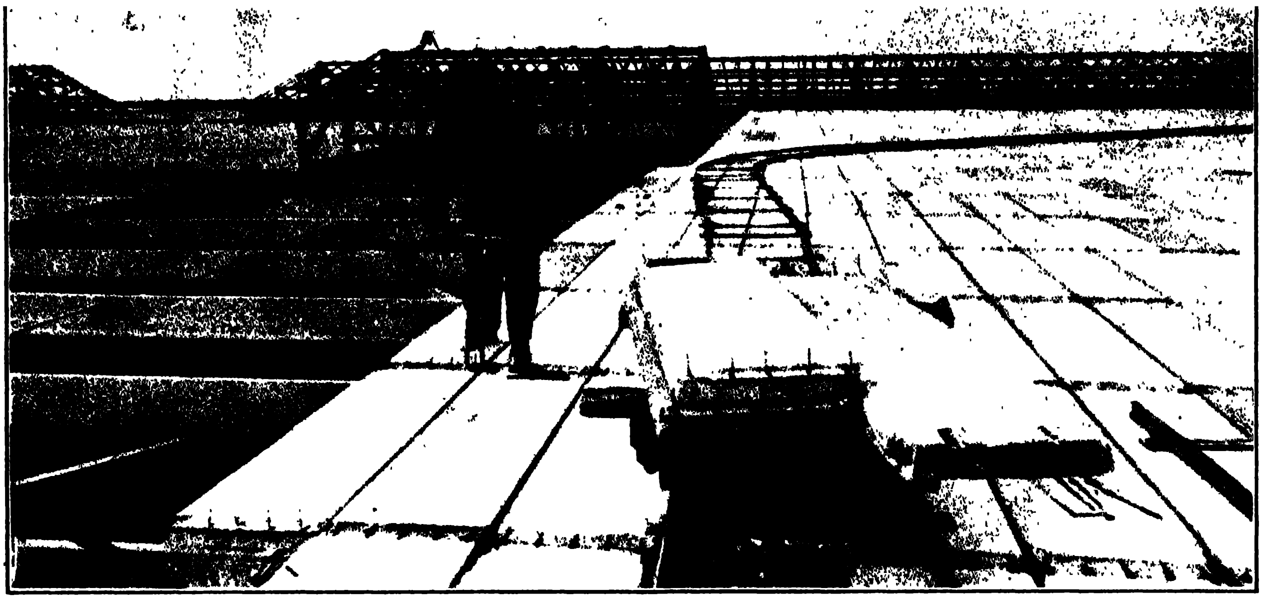


FIG. 264

accepted engineering formulas as illustrated in Table 68. The load capacities of the slab are based upon the structural value of the securely anchored steel cables, in suspension, for a factor of safety of 4. The table applies to the usual range of roof loads. Larger load capacities may be obtained by decreasing the cable spacing. Sizes and load capacities underscored are standards. The wires are designed to carry the load regardless of the compressive value of the gypsum. The most economical spacing of supports is from 5′-6″ to 6′-6″.

Thus it may be seen that the pre-cast slabs are practically the same as those cast-in-place. The pre-moulded is virtually taken apart and put together again at the building. This type is **more commonly used** because of many advantages, some of which are as follows:

(1) maximum insulation is obtained, as the full thickness of the slab extends over the tops of the purlins,
(2) erection is simpler and speedier,
(3) form work is eliminated,
(4) a better under-surface is obtained because of the absence of dripping which is unavoidable with any type of poured construction,
(5) work can be carried on in cold weather, as the grout sets rapidly.

Poured-in-place construction is used when complete fire-protection is desired.*

Many advantages can be claimed for either type of gypsum slab. It is a very good insulator and hence prevents condensation, is of light weight, is a good fire-resistant, is not expansive to any considerable degree, and is fairly elastic.

There are certain **special design considerations** which should be followed because of the nature of the construction. The following are important:

(1) The cost is reduced where the **purlins run transversely to the longest dimension of the building,** as less anchorage is required. This can usually be accomplished even where trusses running across the building exceed the allowable spacing for the slab, by using rafters along the ends and at the peaks of the trusses, into which channel purlins may be framed. In this way, the top chords of the trusses alternate with the channel purlins in acting as supports for the roof slab.

(2) As the peculiar strength of the construction is due to the suspension principle of design, the supporting cables must be securely anchored at their ends. End or **anchorage-purlins** must, therefore, be provided at outside walls, as well as at inside points, such as at bases of monitors, and so on. If anchorage members consist of channels, these should be

* The sections in Fig. 262 are most economical where the purlins are supported by trusses. The encasing of the projecting slabs and flanges of the purlins is usually unnecessary where the truss members are to remain unprotected. The encasement is used where the purlins or beams are supported by girders which may be fireproofed in the same manner.

TABLE 68

TOTAL LOADS

Gypsteel Poured-In-Place — Roof and Floor Construction

Cables composed of 2 No. 12 Cold-Drawn Steel Wires Twisted. Cable = 2 × .00874 @ 20,000 lbs. per sq. in. = 350 lbs. = T

l = Clear Span in inches
20,000 lbs. per sq. in.
Unit Working Stress

$$W = \frac{1152\,T \cdot d}{b \cdot l\,\sqrt{l^2 + 16\,d^2}}\ *$$

W = Total safe load in lbs.
b = Cable Spacing in inches
d = Deflection of Cables — ins

Slab Thickness	Cable Spacing Inches	Clear Distance Between Flanges of Beams													
		3′ 6″	4′ 0″	4′ 6″	5′ 0″	5′ 6″	6′ 0″	6′ 6″	7′ 0″	7′ 6″	8′ 0″	8′ 6″	9′ 0″	9′ 6″	10′ 0″
	1	450	345	273	221	183	154	131	114						
Wt. = 12 lb.-ft.^{2}	$1\frac{1}{4}$	360	276	218	177	147	124	105	91						
	$1\frac{1}{2}$	301	230	182	147	123	103	88	76						
	$1\frac{3}{4}$	258	197	156	126	104	88	75	65						
3	2	225	172	137	110	92	77	65	57						
	$2\frac{1}{4}$	200	153	122	99	82	68	58	51						
$d = 2$	$2\frac{1}{2}$	180	138	109	88	74	61	53	46						
	$2\frac{3}{4}$	164	125	100	80	67	57	48	41						
	3	150	115	91	74	61	52	44	38						
	1	556	430	340	275	227	193	165	141	124	109	96			
Wt. = 14 lb.-ft.^{2}	$1\frac{1}{4}$	445	343	272	220	182	154	132	113	99	87	77			
	$1\frac{1}{2}$	370	285	226	184	152	128	110	95	82	73	64			
	$1\frac{3}{4}$	317	245	192	155	130	110	94	82	71	62	55			
$3\frac{1}{2}$	2	278	215	169	138	114	96	82	71	61	55	48			
	$2\frac{1}{4}$	247	191	150	123	102	85	73	63	54	49	42			
$d = 2\frac{1}{2}$	$2\frac{1}{2}$	222	171	136	110	91	77	66	57	50	44	38			
	$2\frac{3}{4}$	202	155	124	101	83	70	59	52	45	39	35	——	Roofs	——
	3	185	143	113	92	76	64	55	47	41	36	32			
	1	658	508	403	330	270	230	195	169	148	129	115	102	93	83
Wt. = 16 lb.-ft.^{2}	$1\frac{1}{4}$	527	406	322	264	216	184	156	135	118	103	91	82	75	67
	$1\frac{1}{2}$	438	338	268	220	180	153	130	112	98	86	77	69	62	56
	$1\frac{3}{4}$	376	290	230	188	154	131	111	97	84	74	66	59	53	48
4	2	328	254	201	165	135	115	98	84	74	64	57	52	46	41
	$2\frac{1}{4}$	292	226	178	147	120	102	87	75	65	57	51	46	41	37
$d = 3$	$2\frac{1}{2}$	264	203	165	131	108	91	78	67	58	52	46	41	37	34
	$2\frac{3}{4}$	240	185	147	120	98	83	71	61	54	47	42	37	34	31
	3	219	169	134	110	90	77	65	57	49	43	38	35	31	28

Gypsteel Pre-Cast Roof Construction

Cables consisting of $\frac{1}{16}$″ Cold-Drawn Steel-Wire Rods. Cable = .0274 @ 20,000 lbs. per sq. in. = 552 lbs. = T

l = Clear Span in inches
20,000 lbs. per sq. in.
Unit Working Stress

$$W = \frac{1152\,T \cdot d}{b \cdot l\,\sqrt{l^2 + 16\,d^2}}\ *$$

W = Total safe load in lbs.
d = Deflection of Wires - - ins.
b = Cable Spacing in inches

Slab Thickness	Slab Width	Cable Spacing Inches	No. Cables per Slab	Distance Between Supports Center to Center														
				4′ 0″	4′ 3″	4′ 6″	4′ 9″	5′ 0″	5′ 3″	5′ 6″	5′ 9″	6′ 0″	6′ 3″	6′ 6″	6′ 9″	7′ 0″	7′ 3″	7′ 6″
3″ Wt. = 14 lb. per sq. ft.	24″	4	6	136	122	108	98	88	80	72	66	61						
		3.43	7	159	142	126	114	102	93	84	78	72						
	21″	4.2	5									59	54	50	46			
		3.5	6									70	64	59	55			
		3	7									82	76	70	65			
		2.62	8									94	87	80	74			
	18″	3.6	5												54	50		
		3	6												65	60		
		2.57	7												76	70		

* The basic formula for W may be derived by considering the quarter-points and mid-point of the span. The total stress at mid-span, S, is $W \cdot l^2 \div (8 \times 12\,d)$. The total stress at the support in the cables is

$$\frac{S\sqrt{\left(\frac{l}{4}\right)^2 + d^2}}{\frac{l}{4}} = \frac{S\sqrt{l^2 + 16\,d^2}}{l} = \frac{12\,T}{b}, \quad \text{or} \quad \frac{12\,T}{b} = \frac{W \cdot l^2}{96\,d}\,\frac{\sqrt{l^2 + 16\,d^2}}{l} \qquad \text{From this} \quad W = \frac{1152\,T \cdot d}{b \cdot l\sqrt{l^2 + 16\,d^2}}.$$

set with **flanges out so as to** permit of anchors being attached to the top flanges. If the nature of the building requires that these end-channels be set with the flanges turned in, angles should be riveted to the backs of the channels with tops in the same plane.

Where walls extend to or above the anchorage purlin, the inside face of wall must be kept not less than 1" away from the purlin flange for cast-in-place slabs, and 2" away for pre-cast slabs, so as to afford room for placing anchors.

(3) All purlins or other supports for slabs must be set with their **top flanges in the same plane.** On a pitched roof, where purlins rest upon the top chords of trusses, their top flanges will necessarily incline uniformly with the slope of the truss. The eave or end-purlins will, therefore, be framed at the same angle, unless the pitch is relatively slight. If set level, the anchorage-slab will rest upon the edge of the purlin and will not have a full and true bearing upon the entire top flange.

(4) For the same reason, **ridges and the bottoms of valleys should consist,** wherever possible, **of two channels,** framed with flanges facing each other, the top flanges of each in the same true plane as the other purlins on their respective slopes. This permits installing anchorage to each purlin, and such double purlins should be faced so that the projecting flanges are 5" apart at their nearest point. Where roofs are pitched not over 1" to the foot, only one purlin need be used at the ridge.

(5) **With pre-cast slab construction,** where supporting purlins have flanges less than 3" wide, it is necessary to use bearing plates for the temporary support of the slab until the supporting cables are tied up and grouted. **I beam purlins** are, therefore, to be **preferred** to channels, because of their wider flanges, wherever the steel design admits of using the former section economically.

(6) **Tie-rods** should always be specified with pre-cast slabs, as it is practically impossible for the steel erectors to set and hold purlins true to line except by the use of tie-rods.

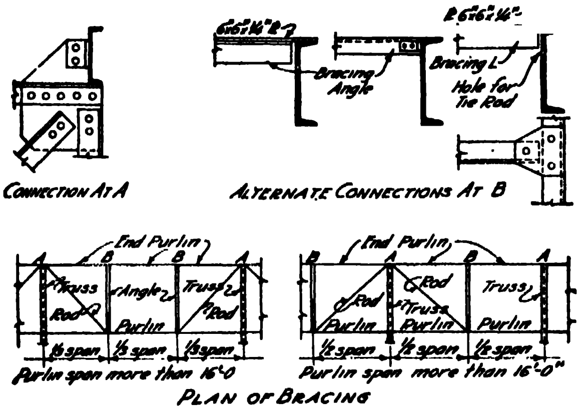


Fig. 265

(7) The supporting steel cables being placed in tension as well as deflection, the resulting pull exerted upon the end-purlins or trusses is provided for by means of light truss bracing as illustrated in Fig. 265. This bracing should be shown on the steel drawings and may be provided by the steel contractor during fabrication at slight cost; whereas, if provided for in a separate contract, the expense of cutting and punching in the field would be substantially greater.

(8) With poured-in-place construction, where the distance between anchorage purlins is four spans or less, it is more economical and quite as effective to omit the angle-and-tie-rod bracing in the end panels, and to have the contractor or owner furnish pipe on the ground in sufficient quantity to provide struts in each panel from one anchorage purlin to the other, which pipe struts will be held in place by clips furnished and set by the erectors. By placing these pipe-struts in each panel, the necessity for the tie-rods in the end-panels is eliminated.

The angle-and-tie-rod bracing is, however, more economical and should be used on all stretches where there are **more than four panels** between anchorage points, and invariably with pre-cast slabs.

(9) **Overhanging Eaves.** With poured-in-place construction, the maximum overhang is 18" beyond the outside edge of the anchorage purlin or truss, except where the slab may pass over and be supported by a wall outside of this purlin or truss, in which case the maximum distance may be 18" beyond the outside face of such wall.

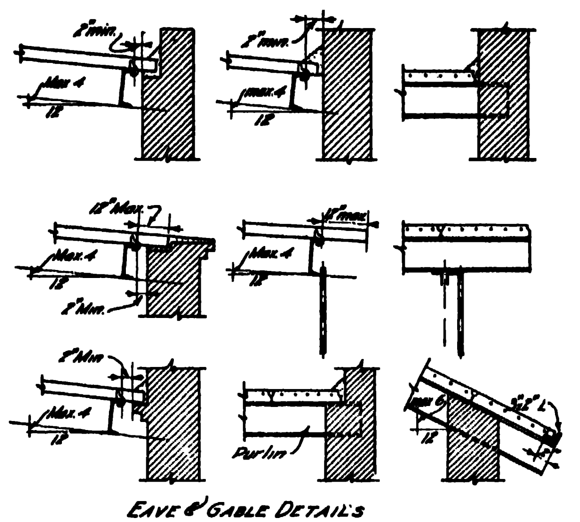


Fig. 266

With pre-cast slabs, the maximum practicable overhang is 12" beyond the outside edge of the anchorage purlin or truss.

Where greater overhangs are unavoidable, outrigger construction as shown in Fig. 266 should be provided.

(10) The **pitch** of the roof should be **limited to ½, or 6" to the foot,** wherever possible, for economical reasons.

(11) **Framing Openings.** With pre-cast slab construction, all openings over 6" diameter must be framed with steel. With poured-in-place construction, it is possible, when necessary, to provide for openings without framing when the dimensions of the same do not exceed 24" **across the cables.** This cannot be done with pre-cast construction, however, as the slab must have both anchorage and support at the edge of the opening. If angles are used for this framing, they must be placed with the horizontal leg **inside** of the opening so as to form flanges properly faced to receive and hold the anchors. In cases where members, angles or channels, at openings receive ends of single slabs, angles should be riveted to these members back to back, to give additional bearing for slabs necessitated by the use of anchor; or I-beams may be used at these points.

(12) **Stop-Angles.** With pre-cast slab construction, where the pitch of the roof exceeds 1–6, or 4" per foot, stop-angles must be framed on the lower purlin or support to take the thrust of the slab.

(13) **Gable and Saw-Tooth Ends.** Gable-ends and the ends of saw-teeth should be built of 3" blocks of Gypsteel composition, laid in gypsum cement mortar, with joints

struck neatly. With this form of construction, the outside surface should be protected with built-up or composition waterproofing.

(14) **Curbs Under and Heads Over Sash.** Curbs and heads should be constructed of 3" solid blocks of Gypsteel composition, which may be reinforced with rods when necessary. The supports for these blocks should be provided by the proper spacing and location of angles or T's, the blocks being laid in place between these steel supports and grouted with gypsum cement mortar, ready to receive waterproofing.

168. Wood Rafters.

For dwellings and for small buildings, small wood beams placed relatively close together, called rafters, may be used to support a wood sheathing (Art. 163), which in turn provides a base for the roofing material. These may also be used when supported directly by the top chords of roof trusses in larger buildings, or occasionally they are supported by purlins between trusses. The spacing is usually made 16", 20" or 24" o.c., depending upon the loads and the spans. In cheap construction the spacing has been made as much as 32" but this is not advisable. The sizes of the rafters required may be calculated in a manner similar to that for floor joists, although data such as given in Table 69 are more commonly employed in their selection. The common sizes which are used for ordinary spans are 2" × 4", 2" × 5" and 2" × 6".*

In some types of construction, separate ceiling frames are used. The only loads which need to be considered for such framing are the weights of the ceiling and joists, and possibly a layer of sheathing above. The basis of design is similar to that of other joists. Table 70 gives the usual limitations for this work.

TABLE 69†

MAXIMUM SPANS FOR RAFTERS

Shingled Roofs not Plastered

Total load, 48 pounds per square foot

Sizes of joists	Distance on centers	Hemlock		White pine		Norway pine or spruce		Douglas fir or Texas pine		Long-leaf yellow pine	
in.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.
2×4	16	7	4	7	9	8	4	9	6	10	10
2×4	20	6	7	6	10	7	6	8	6	8	10
2×6	16	11	1	11	7	12	6	14	2	15	0
2×6	20	9	11	10	4	11	2	12	8	13	4
3×6	16	13	7	14	2	15	3	17	5	18	3
3×6	20	12	2	12	8	13	8	15	7	16	4
2×8	16	14	9	15	6	16	8	18	11	20	0
2×8	20	13	3	13	10	14	11	16	11	17	10
2×8	24	12	1	12	7	13	7	15	6	16	3
2×9	16	16	3	17	3	18	9	21	3	22	6
2×9	20	14	10	15	6	16	9	19	0	20	0
2×9	24	13	7	14	2	15	3	17	4	18	3
2×10	16	18	6	19	3	20	10	23	8	25	0
2×10	20	16	7	17	3	18	8	21	2	22	3
2×10	24	15	1	15	9	17	0	19	3	20	4

* 2 × 5 common in Eastern States only.

† Based upon tables in Kidder's "Architects' and Builders' Pocket Book," John Wiley & Sons, Inc.

TABLE 69 — *Continued*

Shingled Roofs Plastered or Slate Roofs not Plastered

Total load, 57 pounds per square foot

Sizes of joists	Distance on centers	Hemlock		White pine		Norway pine or spruce		Douglas fir or Texas pine		Long-leaf yellow pine	
in.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.
2×4	16	6	9	7	1	7	7	8	8	9	2
2×4	20	6	0	6	4	6	9	7	9	8	2
2×6	16	10	2	10	7	11	6	13	0	13	8
2×6	20	9	1	9	6	10	2	11	7	12	3
3×6	16	12	6	13	0	14	1	15	11	16	9
3×6	20	11	1	11	8	12	7	14	3	15	0
2×8	16	13	7	14	2	15	3	17	4	18	3
2×8	20	12	2	12	8	13	8	15	6	16	4
2×8	24	11	1	11	7	12	6	14	2	14	11
2×9	16	15	3	15	10	17	2	19	5	20	6
2×9	20	13	8	14	3	15	4	17	5	18	5
2×9	24	12	5	13	0	14	0	15	10	16	9
3×8	16	16	7	17	4	18	9	21	3	22	5
3×8	20	14	10	15	6	16	9	19	0	20	1
3×8	24	13	7	14	2	15	3	17	4	18	4
2×10	16	17	0	17	8	19	2	21	7	22	10
2×10	20	15	2	15	10	17	1	19	4	20	6
2×10	24	13	10	14	6	15	7	17	8	18	8

For climates where a 2'-0" snow-fall may be expected. In the southern states, where there is very little snow, spans are safe for slate roofs not plastered, in upper table, and slate or gravel roofs plastered, in lower table.

TABLE 70‡

MAXIMUM SPANS FOR CEILING JOISTS

Sizes of joists	Distance on centers	Hemlock		White pine		Norway pine		Short-leaf yellow pine, spruce		Long-leaf yellow pine, Douglas fir	
in.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.
Total load, 20 pounds per square foot											
2×4	12	8	11	9	3	9	6	9	10	10	7
2×4	16	8	1	8	5	8	8	8	11	9	7
2×6	12	13	5	13	10	14	4	14	9	15	10
2×6	16	12	2	12	7	13	0	13	5	14	5
2×8	12	17	10	18	6	19	1	19	8	21	2
2×8	16	16	3	16	10	17	4	17	10	19	3
2×8	20	15	1	15	7	16	1	16	7	17	10
Total load, 24 pounds per square foot											
2×10	12	21	0	21	9	22	5	23	1	24	11
2×10	16	19	1	19	8	20	5	21	0	22	2
2×10	20	17	8	18	4	18	11	19	6	21	0
2×12	12	25	2	26	0	26	11	27	9	20	11
2×12	16	22	11	23	9	24	6	25	2	27	2
2×12	20	21	3	22	0	22	9	23	5	25	2

169. Roof Purlins

A roof purlin is the member which brings the loads on a roof to the panel points of a truss, as illustrated

‡ Based upon a corresponding table in Kidder's "Architects' and Builders' Pocketbook," John Wiley & Sons, Inc.

in Fig. 256. Consequently, the loads which develop stresses in it are the weight of the roofing and its immediate supporting material, snow and wind loads, and its own weight. It is improbable that the maximum wind and snow loads would act simultaneously for any great length of time, as the wind would tend to blow the snow off of the roof, except in the special case of an alternate thaw and freezing before a high wind. The two common combinations of load which are usually assumed are:

(1) dead load and full snow load, and
(2) dead load, one-half snow load, and full wind load.

One loading may give a maximum pressure perpendicular to the roof while the other may produce a maximum component parallel to the roof, so that both combinations should be investigated for these conditions. The most convenient method is to obtain the resultant pressure per square foot, based upon dead and snow loads acting vertically and wind loads acting perpendicularly to the roof, and then to consider the components of their resultant, perpendicular and parallel to the roof, as illustrated in Fig. 254 (*c*).

The purlins are usually of rectangular wood sections or of rolled steel shapes, depending upon the type of roof to be designed. They generally frame from truss to truss so that they are **considered** to be **simply supported.** The sides of the wood section or the web of the rolled section are usually perpendicular to the top chord of the truss. The direction of the resultant applied load and the vertical principal axis of the member are therefore not coincident, as illustrated in Fig. 267, when the top chord of the truss is inclined. Theoretically, this feature should be considered,* but an approximate solution is sufficiently accurate for practical purposes. The component of the load acting parallel to the roof tends to bend the purlin sidewise and this, as well as the bending caused by the component perpendicular to the roof, must be provided for.

The **common case** in purlin framing is that where the roofing material is carried by plank, slabs and the like, which run over the tops of the purlins. In such instances, these surface materials stiffen the beam in a sidewise direction, and the effect of the component parallel to the roof is assumed to be resisted by them. In other words, the purlins are assumed to be free to bend only in the direction perpendicular to the roof. A rolled steel section, while weak in a sidewise direction, is generally further strengthened by the practical use of tie-rods. Consequently, the usual cases of purlins become those of **simple beam design,** and the **load considered is that in a direction perpendicular to the roof.** When rafters or sub-purlins are used in conjunction with such members, the load is considered to be uniformly distributed, as the concentrations are relatively closely spaced. If jack rafters at larger spacings are used, the procedure involving concentrations should be followed. Figure 267 shows various types of purlin connections.

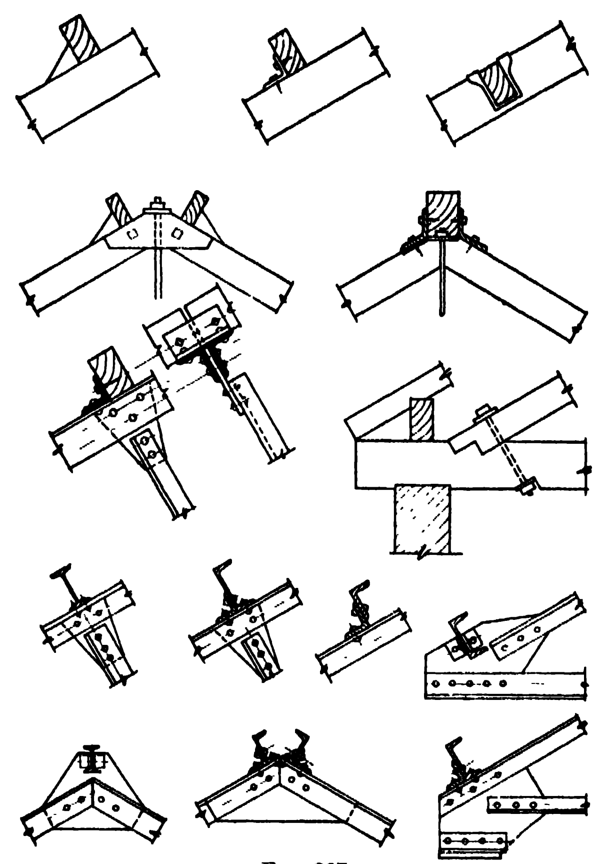


Fig. 267

Illustrative Prob. 169a. What size of purlins is required for a roof with the following specifications:

Inclination of roof 30°. Snow load, 15#/□' of horizontal projection, maximum. Wind load, 24#/□' normal to the roof. Spacing of trusses 16'-0". Spacing of purlins 6'-0". Use rectangular timber sections. 2" plank and slate roof.

Slate = 7	$30 \times 0.866 = 26$#/□' normal component.	
Rfg. paper = 1	(Fig. 261 (*a*) and (*b*).)	
2" Plank = 7		
D. L. = 15		
S. L. = 15 (full snow load)		
T. L. = 30#/□' vertical		
D. L. = 15	$22.5 \times 0.866 = 19.5$#/□' normal component.	
½ S. L. = 7.5		
Sum = 22.5#/□' vertical		

* Refer to the discussion of S-polygons in Hool and Johnson's "Handbook of Building Construction," Vol. I, copyright, McGraw-Hill Book Co., Inc.

$$19.5 + 24.0 \text{ (W. L.)} = 43.5\#/\square' \text{ maximum}$$

$$\text{Load per foot} = 43.5 \times 6 = 271$$

$$\text{Purlin } \frac{6 \times 10}{144} \times 40 = \underline{17}$$

$$288\#/\text{ft.}$$

$$M = 1.5\,w \cdot L^2 = 1.5 \times 288 \times (16)^2 = 110{,}700''\#$$

$$110{,}700''\# = s_w \cdot \frac{b \cdot d^2}{6} = \frac{1500 \times b \cdot d^2}{6}$$

$$b \cdot d^2 = 440 \qquad b = 6, \quad d = 8.4 \qquad \textit{Use } 6'' \times 10''$$

$$D = \frac{5 \times (288 \times 16) \times (16)^3 \times (12)^3}{384 \times 1{,}500{,}000 \times \frac{6 \times (10)^3}{12}} = 0.58''$$

$$D \text{ (allowable)} = \frac{l}{360} = \frac{12 \times 16}{360} = 0.53''$$

Deflection O.K.

A special case occurs when the material carrying the roofing is more or less "flexible" such as corrugated sheeting and various kinds of thin sheathing. The purlin is considered to be free to bend sidewise in such cases. There are two ways of providing resistance to this action, — one, to introduce tie-rods which will offer lateral support, and the other, to proportion the purlin to resist the combined stress developed by the components of the load perpendicular and parallel to the roof, without the use of tie-rods.

Illustrative Prob. 169b. What size of steel [purlin is required for the data of Illustrative Prob. 169*a*, when corrugated sheeting is to be used? No tie-rods are to be employed.

Corr. sheets = 1.5	$22.0 \times 0.866 = 19.0\#/\square'$ ⊥ roof	
Anti-cond. lining = 1.5	$22.0 \times 0.500 = 11.0\#/\square'$ ∥ roof	
Purlins = 4.0		
D. L. = 7.0		
S. L. = 15.0 (full snow load)		
T. L. = $22.0\#/\square'$ vertical		
D. L. = 7.0	$14.5 \times 0.866 = 12.5\#/\square'$ ⊥ roof	
½ S. L. = 7.5	$14.5 \times 0.5 = 7.3\#/\square'$ ∥ roof	
Sum = $14.5\#/\square'$ vertical		

$$12.5 + 24 \text{ (W. L.)} = 36.5\#/\square' \text{ maximum} \perp \text{roof}$$

$$M(\perp \text{roof}) = 1.5\,w \cdot L^2 = 1.5\,(6 \times 36.5) \times (16)^2 = 84{,}000''\#$$

$$M(\parallel \text{roof}) = 1.5\,(6 \times 7.3) \times (16)^2 = 16{,}200''\#$$

$$\text{Try } 10 [\, 15 \quad \left(\frac{I}{c}\right)_{1\text{-}1} = 13.4''^3 \quad \left(\frac{I}{c}\right)_{2\text{-}2} = 1.2''^3$$

$$s_{1\text{-}1} = \frac{84{,}000}{13.4} = 6{,}300$$

$$s_{2\text{-}2} = \frac{16{,}200}{1.2} = \underline{13{,}500}$$

$$19{,}800\#/\square'' \text{ too high.}$$

A 12 [20½ would be required on this basis. The stresses should also be checked for the moments developed by the 19# and 11# components. It should be obvious that the omission of tie-rods requires a much larger purlin section.

The method illustrated above is not the common way of designing purlins carrying "flexible" materials and tie-rods are generally used, as a smaller purlin section may be employed and a stiffer roof results. The stress caused by the moment developed by the component perpendicular to the roof is calculated in the usual way, based upon the span, center to center of trusses. For the component parallel to the roof, the span of the purlin is considered to be the distance between intermediate tie-rods or between a tie-rod and a truss. One bracing member at the middle of the purlin span is usually sufficient (Art. 170). The bending action in a sidewise direction in such instances is then that of a partially continuous beam, supported at its outside points by the trusses and at intermediate points by the tie-rods. Since the support offered by the latter is not as rigid as that given by the trusses, the moment coefficients common to the theory of continuous beams are modified. The bending action is illustrated in Fig. 268. The maximum moment in a sidewise direction on this basis is $\frac{w \cdot L_s^2}{10}$, in which L_s represents the spacing of the tie-rods. The value $\frac{w \cdot L^2}{8}$ is

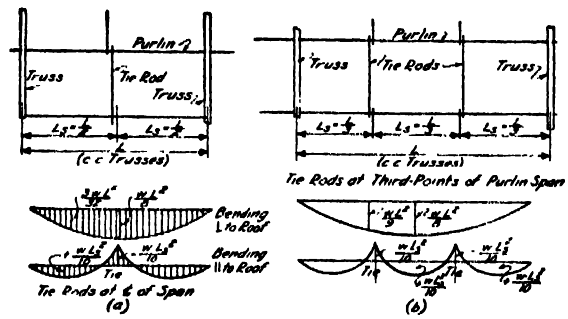


Fig. 268

the maximum moment in a direction normal to the principal axis of the purlin, and L is here the span of the purlin. The maximum combined moment is theoretically the vector sum of the two moments caused by the components and acts in a resultant direction which is nearly perpendicular to the roof.* In practice it is sufficiently accurate to determine the stresses in each direction and add them directly. Such design, however, involves "cut and try" methods.

Illustrative Prob. 169c. What size of purlin may be used for the data of Illustrative Prob. 169*b* if tie-rods are used at the mid-points of the span?

$$M\,(\perp \text{roof}) = 1.5\,(6 \times 36.5) \times (16)^2 = 84{,}000''\#$$

$$M\,(\parallel \text{roof}) = 1.2\,(6 \times 7.3) \times \left(\frac{16}{2}\right)^2 = 3{,}360''\#$$

$$\text{Try } 8 [\, 11\tfrac{1}{4} \quad \left(\frac{I}{c}\right)_{1\text{-}1} = 8.1''^3 \quad \left(\frac{I}{c}\right)_{2\text{-}2} = 0.79''^3$$

$$s_{1\text{-}1} = \frac{84{,}000}{8.1} = 10{,}400$$

$$s_{2\text{-}2} = \frac{3360}{0.79} = \underline{4{,}250} \qquad \text{Use } 8 [\, 11\tfrac{1}{4}.$$

$$14{,}650\#/\square'' \qquad \text{O.K.}$$

$$L = 2\,d \qquad L = 2 \times 8 = 16.0' \text{ allowable}$$

$$L = 16'\text{-}0'' \text{ actual} \qquad \text{Deflection O.K.}$$

$$\text{Pull on tie-rod} = 8 \times 6 \times 11 = 528\# \text{ (see Prob. 169}b\text{)}$$

$$\frac{528}{16{,}000} = 0.033\square'' \text{ net area theoretically required.}$$

Use ¾" tie-rods for practical reasons, namely, to keep the punching of holes standard. By a comparison with Illustrative Prob. 169*b*, the relative economy of using tie-rods should be obvious.

* Refer to discussion of S-polygons, Hool and Johnson's "Handbook of Building Construction," McGraw-Hill Book Co., Inc.

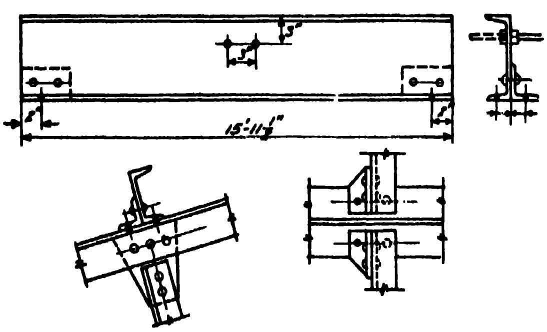
Figure 267 illustrates several methods of fastening both wood and steel purlins to roof trusses of these two materials.

Prob. 169d. What size of purlins would be required in Illustrative Prob. 169*a* if the spacing of trusses were 18′-0″ and the spacing of purlins 5′-6″?

170. Use of Tie-Rods.

Tie-rods are generally used in pitched roofs to brace the steel purlins for practical reasons, irrespective of whether the roofing materials are rigid or "flexible." They not only provide stiffness in a sidewise direction, but they keep the purlins in line during the erection and eliminate the use of extra falsework. When the slope of the roof is slight, say 3″ in 12″ or less, tie-rods are not vitally necessary.

The spacing varies with the length of the bay and the loading. A tie-rod at the mid-span of the purlin is usually sufficient.* In extreme cases of heavy loading or long spans, the tie-rods may be located at the third-points of the purlin span. The size is commonly $\frac{5}{8}''\phi$ or $\frac{3}{4}''\phi$, depending upon the size of the purlin. Theoretically, the size must be sufficient to resist the thrust caused by the component of the wind load, parallel to the roof surface. This is usually an amount which would require only a small rod (Illustrative Prob. 169*c*) and the size as mentioned above is used to make the punching of the holes conform to the rest of the work.



PURLIN DETAILS

FIG. 269

Tie-rods should extend from eave to eave in a continuous line over the ridge so that there will not be an excessive pull on the ridge purlin. They are commonly placed a distance of 3″ down from the top of the top flange of the purlin, and staggered 3″ o.c., as illustrated in Fig. 269, so that the nuts may be turned.

171. Use of Sub-Purlins.

A method sometimes used where slate or tile roofing is employed is that involving sub-purlins. These are simply small steel members running in the opposite direction to the main purlins, which support the slate or tile directly, thereby eliminating a solid form of roofing support such as sheathing, plank, or slabs. This method is illustrated in Fig. 265 (*c*). The trusses are at the usual spacing of 12′-0″ to 16′-0″, and the main purlins are generally about 8′-0″ o.c. This reduces the number of panel points in the trusses and is a factor in determining the type of truss to be used (see Index). The purlins are commonly I-beams or heavy channels. The first method is generally much more economical. The sub-purlins (parallel to the longitudinal walls) are spaced from 8″ to 12½″ o.c., depending upon the roofing material to be carried. These are commonly angles or occasionally small Z-bars (Art. 6). One type of construction is to use 28″ slate laid directly upon Z-bars, 12½″ o.c., with the slate held in place by 2½″ × 2″ × $\frac{3}{16}$″ clip angles. These clips should be bolted nearer one end than the other to increase the leverage of the clip. Anti-condensation linings should be used in such work (see Index).

A special instance of the use of sub-purlins is when **book tile** are employed, as illustrated in Fig. 270 (*a*). In this construction, steel tees (Art. 6) are employed, spaced 1″ farther apart than the

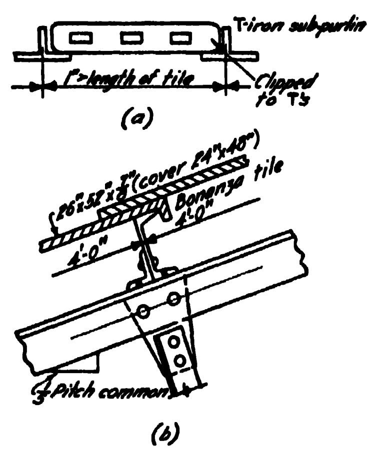


FIG. 270

length of the tile. The latter are usually 2″, 3″ or 4″ thick, 12″ wide, and from 16″ to 24″ long, the 24″ length being common. So-called "government" roof tile are used in a similar manner.

A variation of the usual construction is illustrated in Fig. 270 (*b*), in which Bonanza tile span between channel purlins spaced 4′-0″ apart. Federal cement tile are used in a similar manner.

* When "flexible" roofing is to be used, a theoretical basis for the spacing might be that of lateral support, as discussed in Art. 11.

CHAPTER 14

SIMPLE ROOF FRAMES

172. Definition.

Roof frames may be defined as simple when no trusses are involved, as discussed in Art. 156. They may be either pitched or flat. Of the former, the roofs of houses and other small buildings are usually framed of third-class construction, that is, essentially of wood (Book 1). In framing the roofs of better class residences and occasionally in churches, schools and the like, pitched roofs may be provided with steel rafters and various kinds of enclosure materials. The flat roof is used principally in structures where intermediate supports may be used.

FIG. 271. TYPES OF ROOFS

173. Types of Pitched Roofs.

The particular type of a roof depends upon the shape and size of the plan of the building, the arrangement, use and space desired in the attic, and the design and appearance of the building as a whole, striven for by the architect. Figure 271 illustrates the common types of roofs. The simplest from a standpoint of roof framing is the V or gabled roof. The hip roof, which is an adaptation of the first type, sloping on all sides, is cheaper than the gable roof, considering the saving in gable walls. In this roof the plate is continuous around the building, none of the walls extending above the main plate line.

Gambrel roofs are used in many instances because of the ample attic space they afford, with the lower ridge height. They are especially adapted to wide buildings with finished attics. For the same space, they require less material than the V roof. The Mansard or French roof is similar to the gambrel except that the first slant is steeper and occurs usually on all sides. When a building is over 30′-0″ wide, the deck roof is an economical type of construction. This may be a combination of the Mansard for the front slope and the flat roof, or may have gable ends.

174. Roof Framing Plans.

The preparation of a roof framing plan is more important than has ordinarily been accredited to it. While considerable information may be indicated on the elevations and in the specifications, the cost of making a roof framing plan (and in many cases, a roof architectural plan) will be more than offset by the elimination of errors in the field as to the intention and the location. The lines of the plate upon which the roof is to rest are first definitely located and this framing indicated. The hips, valleys and ridges are next shown. Figure 256 (*a*) illustrates the various types of members in a roof of this kind. Then the common or jack rafters are indicated with the proper spacing for the accommodation of roof openings for the chimney, dormers, or other rising forms. The common rafters forming the trimmers for such openings are usually heavier pieces.

The general method may be illustrated by a reference to Fig. 272. The largest rectangle which can be drawn in this area is *AGEF*. The next step is to draw in the 45° *AH*, *FH*, *CJ* and *JD*, inter-

secting in H and J, respectively. These points locate the ridges of the main rectangle and of the "ell." The lines KL and MN are then drawn showing these ridges. When the two roofs have the same pitch, the intersection of the ell ridge with the main roof may be determined by drawing the 45° line, BO, from the point B. This forms the valleys at the intersection of the ell roof with the main roof.

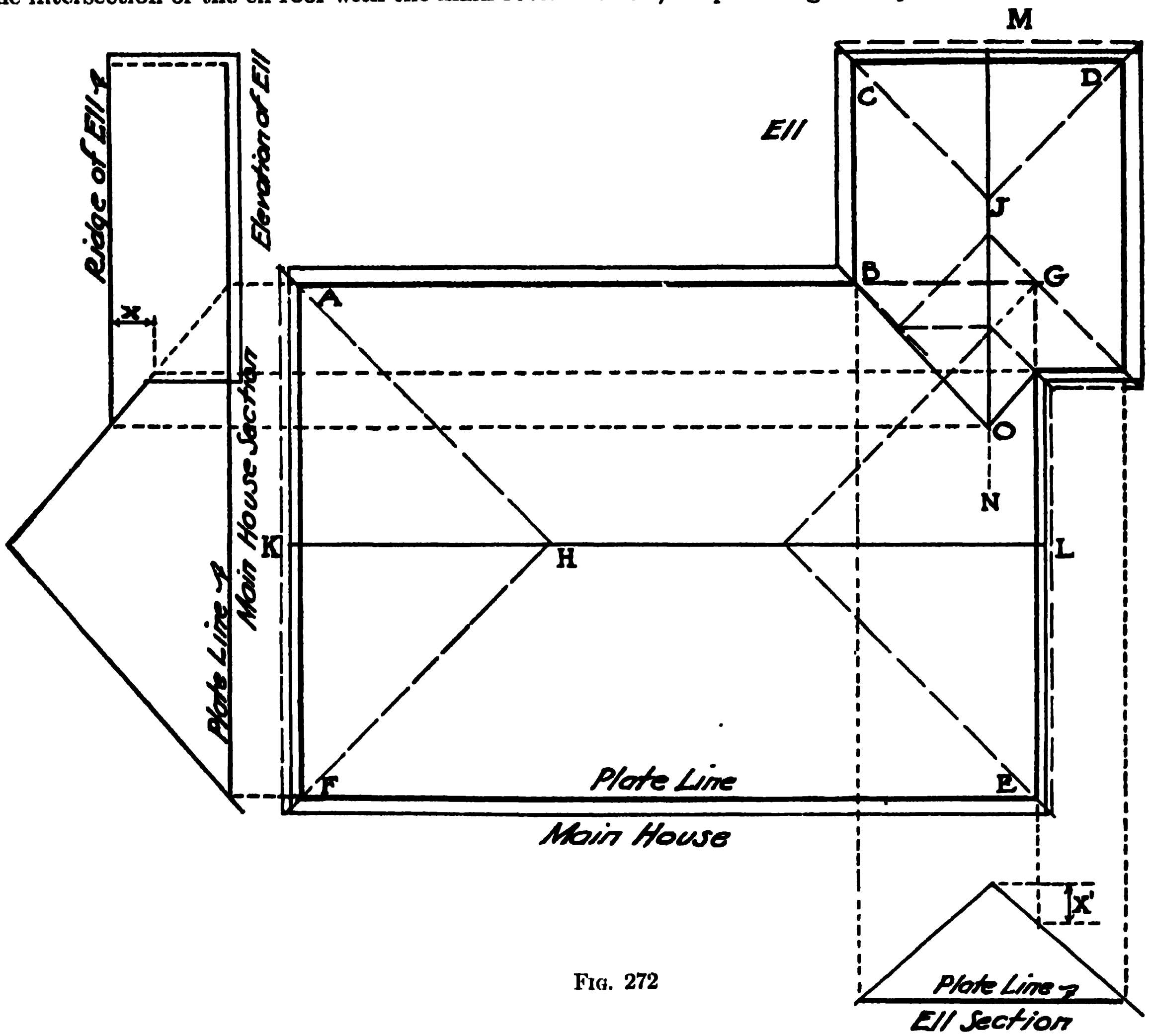


FIG. 272

If gables are used, the roof will appear as indicated by the full lines; if a hipped roof is desired, the roof will be as shown by the dash lines, allowing for projections as indicated. After the general lines of the roof are decided upon, the secondary framing members are drawn in.

Where the pitches of a roof vary, the roof lines must be laid out in conjunction with the elevations and cornice sections. Figure 272 illustrates the methods of projection which are used in connection with the roof illustrated.

In designing hip and valley rafters, it is usually wise to figure the fully tributary areas of load on them (Art. 185). A valley rafter receives such load and is in true position to bend. A hip rafter is of course more or less braced by the jack rafters framing into it each side. If the connections of the jack rafters to the plate are strong enough to resist the outward thrust induced, and the plate or wall, acting as a lateral beam, is strong enough to resist such thrust, a hip rafter generally does not act as a simple beam. However, if there is any appreciable movement outward of the jack rafters at the plate line, this allows the hip member to become more of a beam. For this reason, it is recommended that hip members be designed for the theoretical, tributary loads.

175. Roofs with Steel Rafters.

In certain building zones, wood-framed roofs are not allowed, even for small structures. For such cases, steel rafters may be employed with varying forms of enclosure materials. Instances of this kind also occur occasionally when a better class of construction is desired than is offered by a wood-

framed roof, such as in residences of more imposing nature, churches and schools.

A common form of this construction is that of I-beam rafters supporting reinforced cinder concrete slabs, as illustrated by Pls. 158 and 161, Vol. I. It is necessary to use double forms for such work in order to cast the slabs. The design embodies no new principles of mechanics.

Another variation is to use small purlins of steel tees supporting book tile as illustrated in Fig. 273. The rafters are usually I-beams spaced from 4'-0" to 6'-0" o.c.

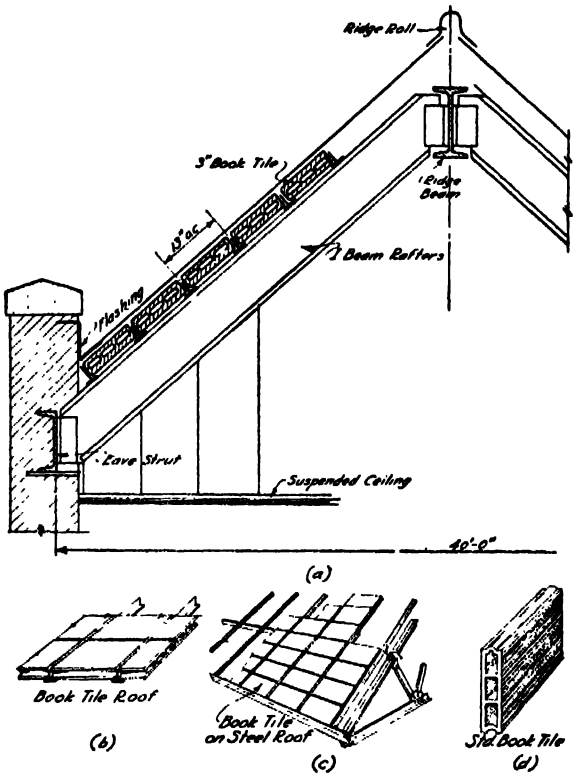


FIG. 273

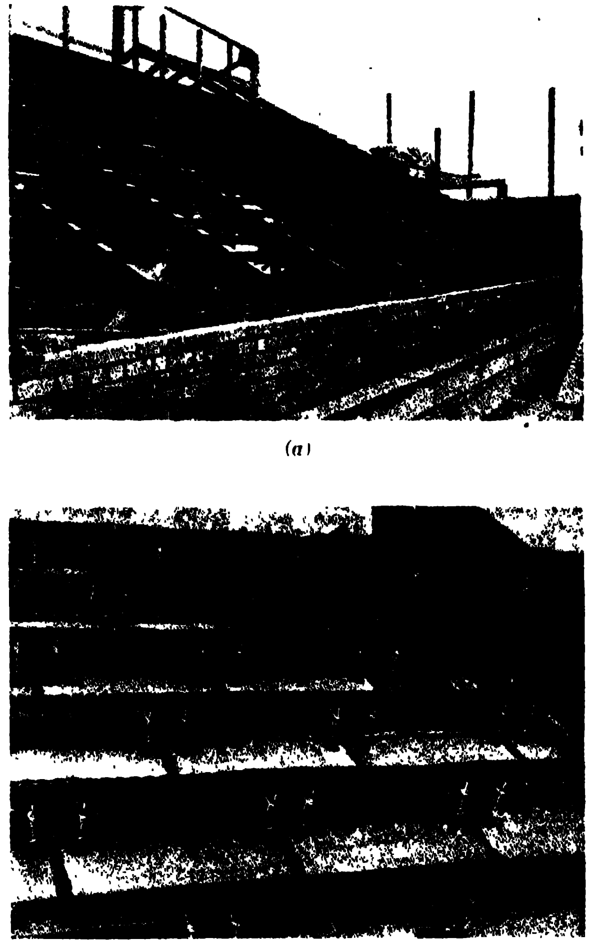
Another variation is shown in Fig. 274 in which steel sub-purlins are used to carry slate directly. That in (*a*) shows the purlins ready, and (*b*) shows a view from the underside with the slate in place. Metal lumber construction (Sect. 8A) is also occasionally used, but it is not easily adapted to pitched roofs.

Prob. 175a. Determine the required sizes of the purlins and rafters for a roof 40' × 64' in plan for the following specifications:

L. L. = 40#/□' of horizontal projection.
¼ pitch roof. 1" flat tile roofing.
Book tile covering 3" × 12" × 24".
Suspended ceiling. Rafters 6'-0" o.c., supported by ridge beam and on wall at the eaves. Ridge beam supported by struts 16'-0" o.c.

176. General Considerations for Flat Roofs.*

In many structures, particularly office buildings, certain types of manufacturing plants, and storage warehouses, it is desirable to have a flat roof. Such a roof is generally defined as one which does not have a pitch of greater than ¾" per foot. The roofing material which may be used in such cases is somewhat limited as classed in Table 60. Tar and gravel or some similar combination, or a "ready roofing" is usually employed.



(a)

(b)

FIG. 274

In the majority of such roofs, the framing is made similar to that of the floor construction, using lighter members because of the smaller loads. The pitch of the roof may be provided in two ways: one, by sloping the framing to obtain the desired inclination; the other, by leaving the frame level as in

* This article is given principally to demonstrate the adaptation to flat roofs. Not all of the possible types of flat roof frames are discussed, but only a sufficient number to illustrate the application of floor construction as discussed in Part II.

floors and producing the pitch with either the roofing material, or, more commonly, by the use of crickets and saddles made generally of cinder concrete fill. There seems to be a tendency to use the latter method except in wood roofs because the cost of the members, increased for any additional weight of fill which must be carried, is generally less than that of the extra labor required in erecting beams and girders to conform to a slight bevel. This is a distinct advantage when future extension is anticipated, and for such a case, the roof frame must naturally be of the same order as the corresponding floor frame. For this type of construction, then, the **design is similar to that of floor construction,** as discussed in Part II, the difference being principally in the amounts of the loads to be carried and the nature of the finish. **Usually a suspended ceiling is employed** in order to provide a dead air space between the roof and the tenants of the top story, which is a distinct advantage both in summer and winter. The height of the space is commonly about 2′-0″. It is naturally an advantage to have the roof construction similar to the floor construction from a standpoint of uniformity of materials and erection methods. However, not all types of floor construction are as readily adaptable to roofs and as commonly used as in floors. In wood framed buildings, plank and timber girders are common in mill construction, and wood rafters and girders for lighter buildings. In steel framed buildings, either slabs or plank and steel beams are common, depending upon the nature of the building. In concrete framed buildings, either beam and girder or flat slab construction are usual, although the various forms of tile-joist framing may be employed. When concrete slabs are used, screeds and sheathing must be provided if the roofing requires nailing.

When it is désired to eliminate some of the columns in the top story to conform with the architectural requirements, such as for dance halls and the like, plate girders, long-span concrete girders, or roof trusses may be used. The trusses may be made to conform to the roof slope in a manner similar to other pitched roof trusses or not, as desired.

For flat roofs, the usual code requirement is that they shall be designed for a live load of 40#/□′. This protects against occasional loads caused by persons walking upon the roof, repair work, and the like, as well as maximum conditions of snow. No wind load obviously need be considered for flat roofs. In cases of future extension, or the use of portions of the roof for recreational purposes, larger loads must be provided according to the particular instances (Art. 90).

177. Use of Steel Beams and Slabs.

The typical frame of structural steel supported by the upper story columns may be used for flat roofs. The beams may support plank or slabs according to the type of building.* Some of the beams may be pitched at sharper angles to conform to special portions of the roof. The outside ends of such beams should be riveted to the wall plates with a sufficient number of rivets to resist the outward thrust.

Pre-cast or poured-in-place gypsum slabs may also be combined with a steel frame (Art. 167).

* An illustration of this kind is given by the roof plan of the City House in Volume I.

CHAPTER 15

GENERAL THEORY OF TRUSSES

178. General Consideration.

Complex roofs, as previously classified (Art. 162), will require trusses. The underlying principles and nomenclature for truss types and solutions is, in the main, the same for all materials. Consequently, the following articles are devoted to a discussion of truss action, the methods of obtaining panel loads, reactions and stresses.

179. Engineering Drawings Versus Details.

In a manner similar to that given for plate girders, the difference between the information relative to trusses which the structural engineer usually furnishes and that which ultimately develops, in the nature of auxiliary details, before the truss is finally erected in its proper position, must be considered. It is also wise to repeat that a good structural engineer should have a working knowledge of details in order that he may better be able to approve drawings submitted to him for his criticism, and also so that he will not design a truss which calls for awkward or perhaps impossible details.

The engineer's layout usually shows only the general outline of the truss, the stresses in the members and the required sizes of them. The first includes information relative to the span (center to center of bearings), the rise of the truss, the number of panels and the distances between panel points, and so on. In general, the diagram must show the direction and relation of all the members. A common custom is to indicate the stresses and loads on one side of the center line and the sizes on the other side, for corresponding members. Some engineers often give additional information such as the important details, splice locations, and the like, in order that their intentions will not be misconstrued. It is always wise to indicate the location and size of all bracing members used in the plane of the top chord, in the plane of the bottom chord, and such side or transverse bracing as may be necessary (Art. 198.)

The details are usually supplied by the contractor who has competed for the work successfully. These must include the location of all working points, such as the intersections of the working lines of the members, the lengths between all working points, and so on. All dimensions should be limited in their minimum accuracy to $\frac{1}{16}''$. The inclinations of the web members are referred to horizontal or vertical directions, usually by indicating the rise or run in $12''$. This may be given to the nearest $\frac{1}{16}''$, but not any closer. It is also necessary to completely define details such as those at the joints, splices, connections for bracing and the like, which are required to build the truss. Generally two scales are used on the same elevation, one to show the outline of the truss and its main dimensions, and the other to indicate the details. The former is commonly $\frac{1}{8}''$ or $\frac{1}{4}'' = 1'\text{-}0''$, and the latter $\frac{3}{4}''$ or $1'' = 1'\text{-}0''$. Plate 29 represents a typical drawing of this kind.

180. Definition and Conception of Truss Action.

A truss is a rigid, jointed framework usually supported only at its ends, and designed to act as a beam in transferring loads to the supports. It is therefore subjected to the usual reactions, shears and moments, but the members are commonly arranged so that each is transmitting longitudinal stress only, — either tension or compression. A

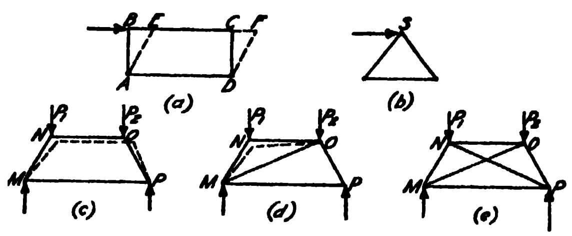


FIG. 275

truss should form an unyielding structure composed wholly of rigid triangles, in order that there shall be no tendency toward distortion. To illustrate, if a force is applied to an apex of a quadrilateral, such as $ABCD$ in Fig. 275 (*a*), the latter would tend to assume the shape indicated by the dotted lines $AEFD$. In contrast, the shape of the triangle RST in Fig. 275 (*b*) remains fixed to the extent that it cannot be altered without changing the length of the sides. The trapezoid $MNOP$ in (*c*) is in equilibrium only under a particular dis-

tribution of loads, namely when P_1 and P_2 are equal, and may be called "imperfect" for this reason. It would tend to change its shape when the loads varied. A group of triangles may not always be "perfect," however. Thus in Fig. 275 (d), when the diagonal MO is introduced, two triangles result but the disfiguration could occur, and the frame would be imperfect if MO were incapable of resisting compression. If the other diagonal were introduced, namely NP, together with MO, the frame would become statically indeterminate, that is, it is self-deformed, and the stresses in the diagonals cannot be obtained by the ordinary theoretical methods of statics without making the assumption that either diagonal may act in tension, as required. Such members are sometimes called "semi-members" or redundant members, because one diagonal is stressed for a given condition of loading, whereas for some other case of loading, the other diagonal is stressed. **Redundant members** are often classed as those in excess of the minimum required number to make a rigid structure when each member is capable of carrying either tension or compression. When a member may be stressed in tension due to one condition of loading and in compression due to another instance of loading, it is usually called a **counter member,** or counterbrace.

As mentioned above, the action of a truss is to transfer the applied loads to the supports. Any given load is carried from joint to joint by the nearest chord and web system to the support and thence eventually transferred to the ground. Thus in Fig. 276, the load P is carried partly to R_1 and partly to R_2. The first may be conceived as

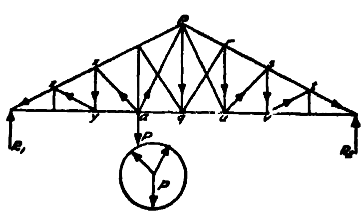


FIG. 276

being carried up to x, down to y, up to z, and then down to R_1. The portion of load transferred to R_2 follows a path of a–p–q–r–u–s–v–t–R_2.

A truss is not weakened by its lack of symmetry or by a variably sloped bottom chord, providing all the members are strong enough themselves to carry the stresses. In Fig. 277, there is an assemblage of adjacent triangles each having a common side, thus conforming to the definition of a truss. Symmetry is, however, desirable for many reasons among which appearance, economy of fabrication and details are important. A true truss does not depend upon the rigidity of its joints for its stability. Furthermore, when the reactions are vertical, they may be determined by statics and the frame is said to be a "simple truss." When the loads are inclined, assumptions relative to the directions of the lines of action of the reactions must be made in order to keep the investigation within calculable limits. When the reactions are inclined even under vertical loads, they cannot be obtained by simple statics, and the truss is theoretically an "arch."

Prob. 180a. Make a sketch of a 6 panel Fink truss, show loads at the top chord panel points and one at the bottom chord at mid-span. Trace how these loads are transferred to the supports.

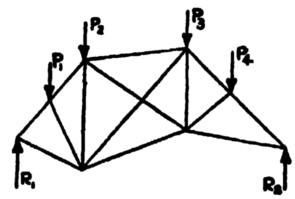


FIG. 277

181. Component Parts of Trusses.

In order that the ensuing discussion may be more clearly understood, Fig. 278 is given, showing the names of the component parts of a truss. A **bay,** such as $AMNC$, is usually defined as a volume and includes all of the material between corresponding

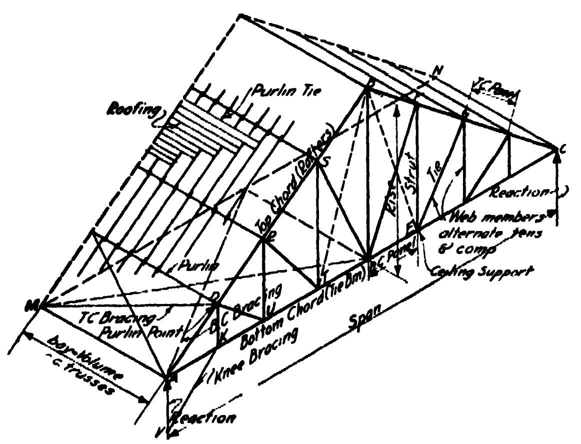


FIG. 278

supports of the trusses for a given length of the building. This distance is often termed **center to center of trusses** (c.c. trusses), as AM. The distance between the supports of a truss is defined as its **span.** This is taken as the distance center to center of bearings when the truss rests upon walls or runs over the tops of columns. When it frames in between columns, the span is usually taken as the distance face to face of columns.

The various parts of a truss are called **members.**

The member forming the shape of the roof is called the **top chord** (T.C.), such as AB and BC in Fig. 278. The lower members, which are commonly horizontal, constitute the **bottom chord** (B.C.), as AC. The remainder is called the **web-system,** such as DK, DU, UR, and the like. Members such as DK and RU are sometimes named **uprights;** and DU and RT, **diagonals.** On a basis of stresses, compression members are called **struts;** and those carrying tension, **ties.** It is convenient to remember that in the majority of trusses, the web members are alternately compression and tension members. A member such as DK is sometimes specially defined as a **sag tie.** Theoretically no stress exists in such a member, but its purpose is to prevent excessive deflection of the bottom chord member, AU, particularly when the latter is unusually long.

The intersection of two or more members is called a **joint.** The opposite may be stated,-- that a member connects any two adjacent joints. **Main members** are those which act when the entire structure is loaded, and **counter members** are those which act only for particular loads, such as BT in Fig. 278. A **counterbrace** is a member which is designed to resist either tension or compression in turn. A tension member may resist some compression and not necessarily be a counterbrace if the compression is sufficiently below the value of the tension. The portion of a truss which lies between two purlin points, such as D and R, or between two ceiling load joints, as K and U, is called a **panel.**

Trusses are often braced in the chord planes. This is called **top chord bracing** or **bottom chord bracing,** depending upon the chord plane it occurs in. Occasionally trusses are stiffened at columns by using **knee braces,** KV.

182. Selection of the Type of Truss.

The selection of the type of roof truss to be used in any particular instance depends upon many factors. These may be summarized as follows:

(1) the **contour** of the roof surface, such as its pitch and the nature of the roof supporting frame,

(2) the **shape of** the roof **void,** which involves the available height for the truss, the allowable headroom, and so on.

(3) the limits of the possible **arrangement in plan,** such as the span, spacing of trusses, manner of support, and the like,

(4) **special loads** such as those due to machinery and equipment,

(5) the simplicity and **appearance** of the truss,

(6) the question of whether the truss is to be open or exposed in the upper story, or whether there is to be a **ceiling.**

(7) the relative **economy** of one type compared with another,

(8) the **material** to be used for the truss, and

(9) the **character of** the **building,** whether it is to be public or residential, mill, shop or monumental, permanent or temporary.

The architectural considerations have much to do with the possible types of trusses which may be used, and the first six factors given above are principally determined by the architect's plans. Furthermore, only certain types of trusses are adaptable for a given material. For this reason, types of trusses are discussed here only in a general way and particular applications are taken up later. The type of the building is also important, as certain types are almost invariably used for a given kind of structure, based upon previous experience. The contour of the roof is probably the most important factor, and trusses are naturally divided into types upon this basis. The following classification may be used:

(1) V-roofs, sometimes called pitch or gable roofs,

(2) Flat roofs,

(3) Gambrel, mansard and deck roofs,

(4) Curved or arched roofs,

(5) Domical roofs,* and

(6) Special trusses, such as those supporting balconies, those in grandstands, sheds, and floor construction, and cantilevers.

Almost any combination of triangles may be made to support a roof, but some arrangements will be more economical than others. In general, the members should be well placed with respect to the loads. From the standpoint of economy, it is desirable to have all loads brought on to the truss at panel points. This means that the same general type of truss may be used for spans from 40′-0″ to 80′-0″ or larger, by simply varying the number of panels (Art. 183). A horizontal bottom chord is usually preferable, particularly when the top chord is inclined at an angle of 30°, or less. When appearance or headroom controls, a raised top chord is used. This is an advantage as far as the shorter lengths of the web members are concerned, but the stresses in them are slightly increased, due to the shallower truss depth. Tension members of excessive length should be avoided, as considerable stresses are induced by their own weight. Compression members should be kept as short as possible to reduce the tendency towards sidewise bending.

* Considered in Chap. 19.

Figures 279 and 280 illustrate several types of trusses.

183. Panel Widths.

As previously stated, the same form of truss may be used for widely varying spans by changing the number of panels in the truss. The panels which a truss may be divided into depend upon the allowable span of the materials supporting the roofing,

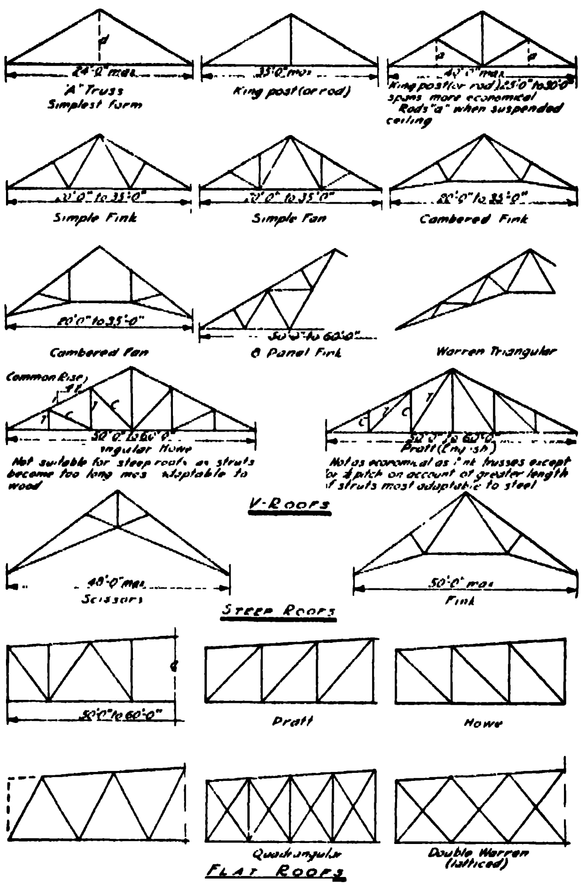


FIG. 279

their arrangement, and in some cases, upon the type of truss selected. There are four common methods used to frame a trussed roof, namely:

(1) A series of roof joists is supported directly upon the top chord, as in Fig. 281 (*a*).

(2) Purlins supported only at panel points, with sheathing or other materials (or rafters), paralleling the top chord, as in (*b*), may be used.

(3) The use of heavier purlin sections and more widely separated panel points, with jack rafters framing between the purlins. Sheathing or other materials are used to span over the jack rafters, as in (*c*),

(4) The use of small purlin trusses or latticed girders at certain panel points with heavy rafters framing between them, as in (*d*).

The second method is by far the most common. There are advantages and disadvantages in each

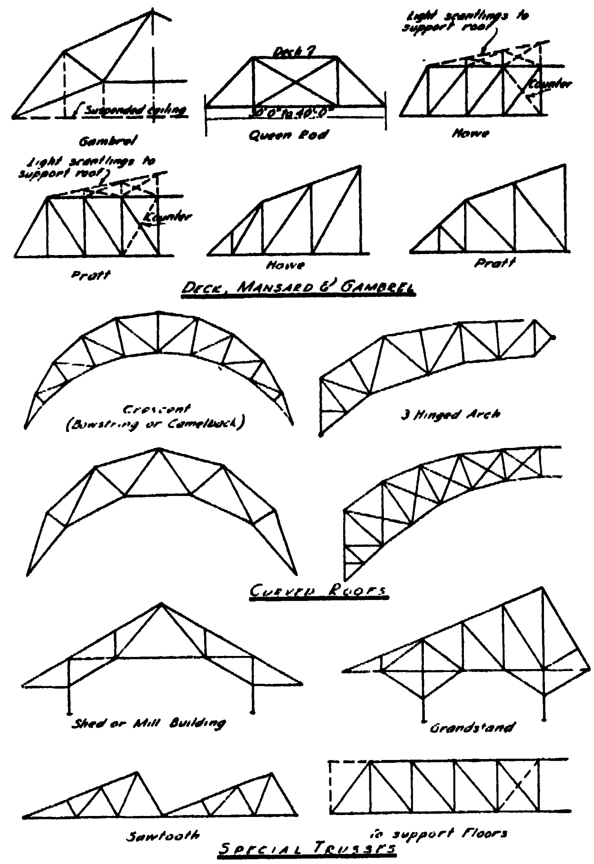


FIG. 280

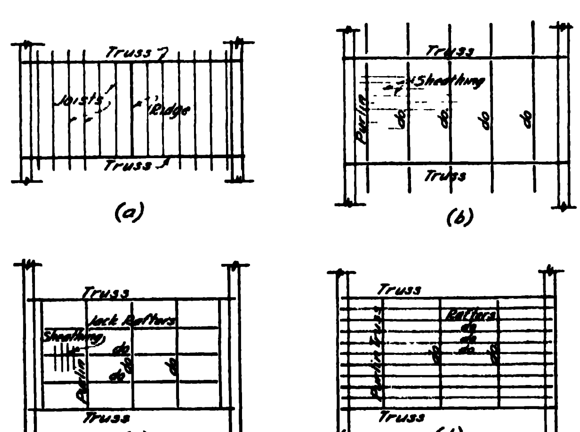


FIG. 281

system, but the cost in each averages about the same. It is desirable to locate a panel point at each concentration of load brought to the truss, as this arrangement is usually more economical, inasmuch as

the truss members are subjected to direct stresses only. When loads are brought to a truss between panel points, indirect stress due to bending in addition to the direct stress must be provided for. (Art. 196.) This is economically possible only when the increase in the section of the members is offset by the saving in the weights of the purlins.

An important reason for maintaining a relatively close spacing of panel points lies in the desirability of keeping the inclination of the diagonals of a truss between 30° and 60° (45° most desirable). It is difficult to frame web members into the chord members when the inclinations are sharp. In special cases, the location of some purlins may be established by the requirements of skylight or monitor framing, or the like. The other purlins should then be arranged to divide the remainder of the spaces equally. It is a distinct advantage to keep the panels symmetrical about the center-line of the truss span, if possible, on account of the simpler details and fabrication involved. The following represents good practice for ordinary cases:

Panel Point Locations

Framing	Average	Maximum
Regular purlins	6′-0″	8′-0″
Jack rafters and purlins	10′-0″	12′-0″
Joists directly on top chord	4′-0″	5′-0″

184. Spacing of Trusses.

The distance between trusses is primarily controlled by the location of the supports, particularly when columns, piers, or pilasters are used. When the ends of a truss rest upon regular bearing walls, the limitations which window openings and other architectural details impose must be considered. The location of supplementary equipment, such as shafting, heating apparatus, cranes, electric conduits, balconies, platforms, water or compressed air tanks, lifts and skylights, is also very significant.

There is an economic arrangement for every roof frame. Theoretically, shorter distances between trusses will effect less total weight for the trusses and purlins per square foot of roof covered. However, a large number of trusses increases the total cost of fabrication. If the spacing of the trusses is increased, the weights of the purlins and bracing are increased, while those of the trusses and columns, if any, per unit area are decreased. The cost of the purlins varies about as the square of their span. This, however, is based upon a lower unit cost than that of the trusses on account of the lesser amount of fabrication required. The roofing and the roof covering materials are directly proportional in cost to the area covered, but this unit cost will usually be less than that of the purlins. The problem in any case is one of keeping the sum of the costs to a minimum, and this depends upon comparative unit prices as much as upon practical limitations.

A small change in the spacing of the trusses has little effect upon the total weight of the roof, and the factor which usually decides this is the common range of commercial sizes for purlins. In order that these may be safe for flexure and deflection, their spans must be limited. For stock sizes, the spans range from 12′-0″ to 25′-0″, 20′-0″ being the average. In very large roof frames, latticed purlins may be used in order that the spacing of the trusses may be in proportion to their spans, and in such cases, the spacing may be increased up to 40′-0″, or even 50′-0″. Generally, this adds weight to the roof as a whole, and is not commonly resorted to unless the larger bays are warranted.

In summary, guides have been developed which represent reasonable values based upon the usual spacings of supports, economic considerations, and limiting spans of stock purlin materials. A rule of thumb is to space trusses which span from 40′-0″ to 200′-0″ about $\frac{1}{4}$ of their span. For the shorter spans, $\frac{1}{4}$ is used quite commonly, while $\frac{1}{5}$ is more customary for the larger spans. In the shorter spans, there is a tendency toward wider spacings, but 16′-0″ to 20′-0″ conforms best with the usual spacing of columns and the like. Since the spacing is a function of the weight to be carried, it should be based upon the truss span.* The following represent good practice in this regard, and comparative designs† have verified the correctness of the values:

Truss Span	Spacing of Trusses
20′-0″ to 30′-0″	12′-0″
30′-0″ to 60′-0″	16′-0″
60′-0″ to 80′-0″	20′-0″

185. Panel Loads.

One of the most important steps in the design of a truss is the calculation of the correct amount of load which is carried to each panel point. If such values are not established to within reasonably accurate limits, the calculations for the stresses in the truss members, and also their subsequent application to the design, are of little value. The panel loads are usually calculated separately (except in cases of combined loading, Art. 161), that is, as dead panel loads (D.P.L.), live panel loads (L.P.L.) and wind panel loads (W.P.L.). In this manner, maximum combinations of stress are more easily determined.

The loads per square foot of roof surface have been previously discussed (Chap. 12). The calculation of panel loads is then simply a determina-

* A truss resting upon masonry walls is probably stiffer than one resting upon columns, so that, in general, trusses may be spaced a little farther apart in the first case than in the second.

† See Bulletin No. 16 issued by the Engineering Experiment Station, University of Illinois.

tion of the number of square feet of roof surface tributary to a panel point, at so much unit load. In general, this area is the result of the product of the distance between panel points and the spacing of trusses. The customary procedure is to make an estimate for the weight of the truss itself, as so much per square foot of surface covered, and to add this to the other dead load values, and apply it with the other dead panel loads (usually on the top chord). This obviously introduces slight errors but these are insignificant as a rule. The following illustrates the application of this discussion to the panel loads for a simple roof.

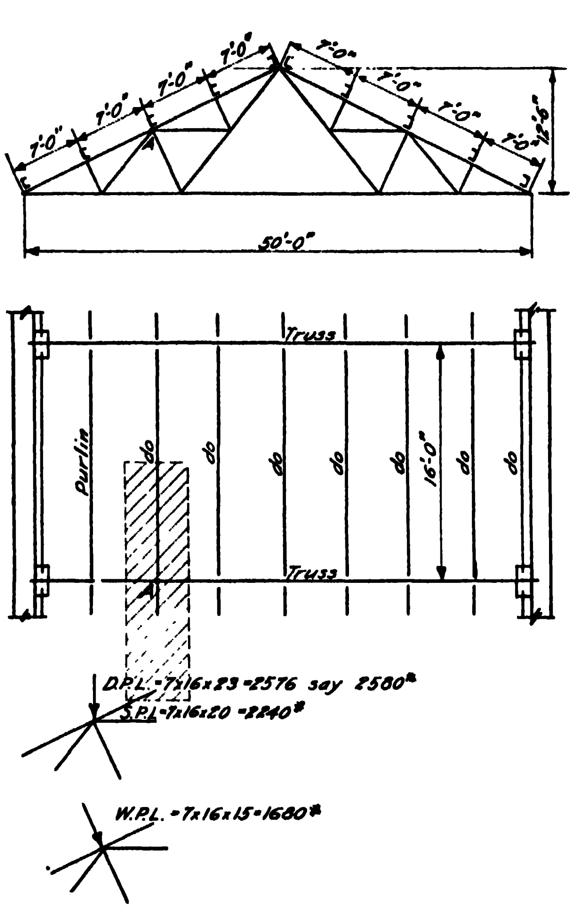


FIG. 282

Illustrative Prob. 185a. Determine the panel loads for the trusses shown in Fig. 282, for the following specifications:

Snow load 20#/□'. W.L. 15#/□' normal to roof.
Slate roof, 2" plank sheathing, steel purlins, steel trusses 16'-0" o.c. ¼ pitch roof.

Slate	6
2" Plank	7
Purlins	4
Trusses	6
T.D.L. =	23#/□'

Loads at panel point A.

7 × 16 = 112□' of roof
112 × 23 = 2580# D.P.L. (Vertical)
112 × 20 = 2240# S.P.L. "
112 × 15 = 1680# W.P.L. (⊥ Roof).

In **hipped roofs,** the tributary areas may be calculated in a similar manner, although the work involves more complicated computations. In steep roofs, the hip probably does not actually receive its full share of the load due to the trussing action of the jack members. For valley members, the actual load may possibly exceed the theoretical value as they support the bottoms of the roof frame at such points and there is a tendency toward a downward sag already, due to the relative positions of the members. For this reason, valley members should always be designed conservatively, as there is a tendency toward downward thrust. However, the loads are assumed to be transferred in a manner similar to horizontal beams. Designing hip members as beams for the full tributary area often gives results which may seem excessive when the trussing action is considered. The jack members, however, will not supply their full trussing action unless they are secure against movement at their bottom points. Thus they should have stiff connections at the bottom to supply resistance to their thrust or the lateral support should be checked to be safe in bending.

Illustrative Prob. 185b. Determine the number of square feet tributary to joints A, B, C, and D in Fig. 283.

Load at A = reactions of hips SA and DA, and area of rectangle $RAPQ$, rectangle $LMNA$, and portions of triangles ATC and ABU.

Hip SA carries triangles SRA and SLA.

$$SA = \frac{6 \times 9}{2} + \frac{6 \times 9}{2} = 54\square'.$$

The center of gravity of the two triangles may be found by drawing the medians.

$$\text{The load at } A = \frac{54 \times SV}{SA} = \frac{54 \times 89}{153}$$
$$= 31.4\square' \text{ from } SA.$$

The load on DA is the same as on SA.

The load at A = 54.0 − 31.4 = 18.6□' from DA.

Rectangles $RAPQ$ and $LMNA$ = 6 × 4.5 = 27□' each.

$$\text{Purlin } AC \text{ supports } \triangle ACT = \frac{6 \times 9}{2} = 27\square'.$$

$\frac{1}{3}$ transferred to A = $\frac{1}{3}$ × 27 = 9□'.

$$\tfrac{1}{3} \text{ of } \triangle ABU = \tfrac{1}{3} \times \frac{6 \times 9}{2} = 9\square'.$$

Summary

From hip SA =	31.4
DA =	18.6
$RAPQ$ =	27.0
$LMNA$ =	27.0
$\frac{1}{3}\triangle ACT$ =	9.0
$\frac{1}{3}\triangle ABU$ =	9.0
	126.0□' *Total at panel point A.*

Load at B.

One side $= \frac{2}{3}\,\triangle ABU + NMWB$

$$= \tfrac{2}{3} \times 27 + \frac{6 \times 9}{2} = 45\square'.$$

Total load at panel point $B = 2 \times 45 = 90.0\square'$.

Load at C.

$\frac{2}{3}\,\triangle ACT$ + area $PQXYTCP$

$\frac{2}{3} \times 27 + 3\,(6.0 \times 4.5) = 99.0\square'$ *Total load at panel point C.*

Load at D = [load from hip $AD + TYZD$] $\times 2$

$= [31.4 + 6.0 \times 4.5] \times 2 = 116.8\square'$ *Total load at panel point D.*

The load at E is the area $XGHY = 12 \times 9 = 108\square'$.

The load at F is the area $YHIZ$ doubled $= 108\square'$, obtained as in a simple roof.

Prob. 185c. What dead load per sq. ft. should be used in designing a truss for the following conditions:

1″ Tile Roofing. 2″ Cinder Fill. 4″ Cinder concrete slab. Steel purlins.

What is the dead panel load if the purlins are 6′-0″ o.c. and the trusses are 15′-0″ o.c.? Roof ¼ pitch. What is the wind panel load for a pressure of 30#/□′ on vertical surfaces? What is the snow panel load at 20#/□′?

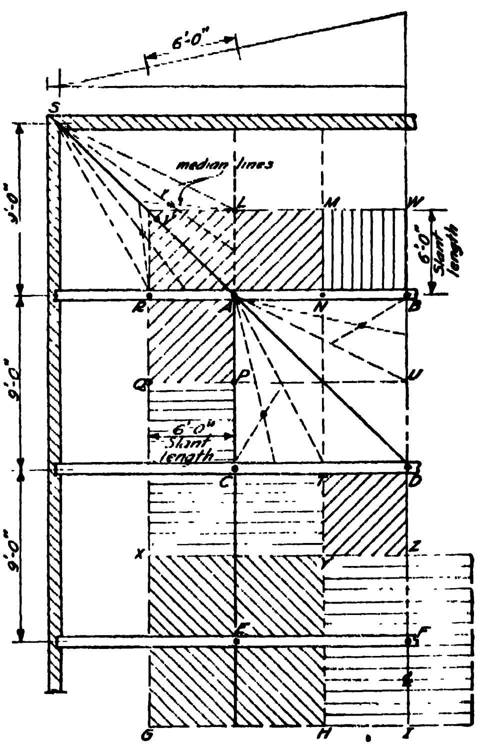


FIG. 283

186. Reactions Due to Vertical Loading.

When the arrangement and the values of the loads on a truss have been established, as previously discussed, the next step is to obtain the reactions at the supports, caused by the loads. The usual procedure in truss design is to establish the values of the reactions caused by the dead loads, snow loads, and wind loads separately.* Truss reactions may be determined by either analytical or graphical methods. The choice of the method depends upon the type of the truss and the arrangement of the loads, but in general it may be said that the analytical method is more commonly used to obtain the reactions of simple roof trusses. In complex trusses with unusual loads the graphical method will be found more economical of time.

In the **analytical method,** the principles used in calculating beam reactions may be employed, since a truss may be considered as a beam. When the loads on a truss are symmetrically arranged with respect to the span, there is no need of any moment calculations (or graphical solution either) to determine the values of the reactions. They are each equal to one-half the total load on the span for simple trusses and the values may be determined by inspection. Thus in Fig. 284 the reactions are as follows:

$$R_1 = R_2 = \frac{1500 + 5\,(3000) + 1500}{2} = 9000\#.$$

These values may be computed by the method of moments in a manner similar to that for unsymmetrical loads, but once the truth of such reasoning is established, this work becomes unnecessary. The majority of vertical loads on roof trusses are symmetrically arranged, and the matter of calculating vertical reactions is a simple procedure.

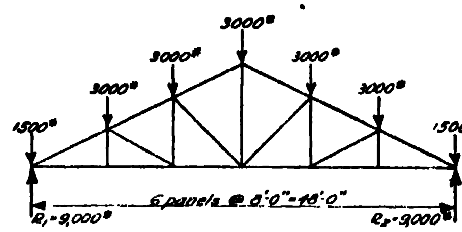


FIG. 284

In special cases unsymmetrical loads may occur, such as those from balconies, shafting, and so on. The reactions in such instances cannot be determined by inspection, and a solution by moments or graphics is necessary. Bearing in mind that the moment of a force is the force times its perpendicular distance from the moment center to its line of action, the forces in Fig. 285 (a) may be imagined as acting like those in (b), as far as their rotation about a support is concerned. This resolves itself into nothing more than calculating beam reactions. Applying this method in Fig. 285,

* Wind loads are considered to act normal to the roof. The reactions caused by them are discussed in Art. 187.

$$\Sigma M_A = 2400 \times 6 + 6000 \times 10 + 3000 \times 15 + 4000 \times 23 + 2000 \times 31 - 31\,R_2 = 0$$

$$R_2 = 8800\#$$

$$\Sigma P = 18{,}400\# \qquad R_1 = 9600\#$$

In a similar manner, the reactions for other irregular trusses or cantilever trusses, and so on, may be established.

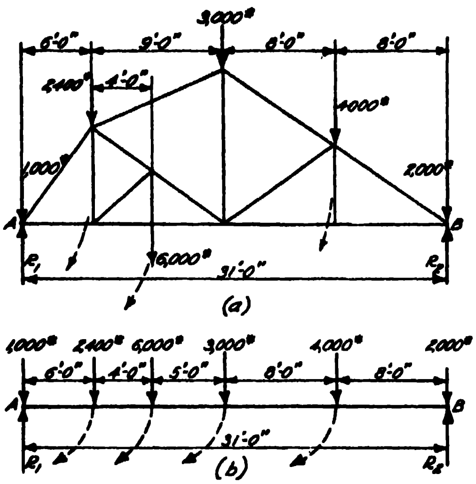


FIG. 285

A **graphical method** for the determination of truss reactions may be used when a graphical solution for the stresses is anticipated. Usually however, the analytical solution is quicker and more accurate, and the results may be incorporated in the graphical layout for the stresses. The former is very similar to those used to obtain the reactions for a simple beam by graphical construction.

The method may be illustrated by a reference to Fig. 286. A diagram of the truss at a convenient scale (lengths) as in (*a*), is first drawn (Space Diagram). Particular attention is called to the manner in which the spaces between the loads are lettered. These occur in alphabetical order from left to right, using capital letters. The space below the bottom chord is designated by Y in all cases except when loads are applied to the bottom chord, when the spaces may be designated as X, Y, Z, etc. A convenient scale (force units) is selected to represent the loads and these are plotted, as the line aj in (*b*) (Force Diagram). The line may be started at any convenient point. The representation of the loads is accomplished by labeling the extremities of the scaled length with the lower case letters corresponding to the capital letters in the spaces either side of the load. Thus, going from space A to space B in the truss diagram, the 1000# load is passed, which is graphically represented by ab on the load line, that is, the force between A and B is a downward force a to b, and so on. The loads are plotted by proceeding in a clockwise direction around the truss. Less confusion results if the load at the left hand end of the truss is first plotted.

When the load line is completely drawn to scale, representing all the loads on the truss, the next step is to select a **pole,** P. This may be any point in the plane of the paper, but naturally it is desirable to select a convenient position. Lines are then

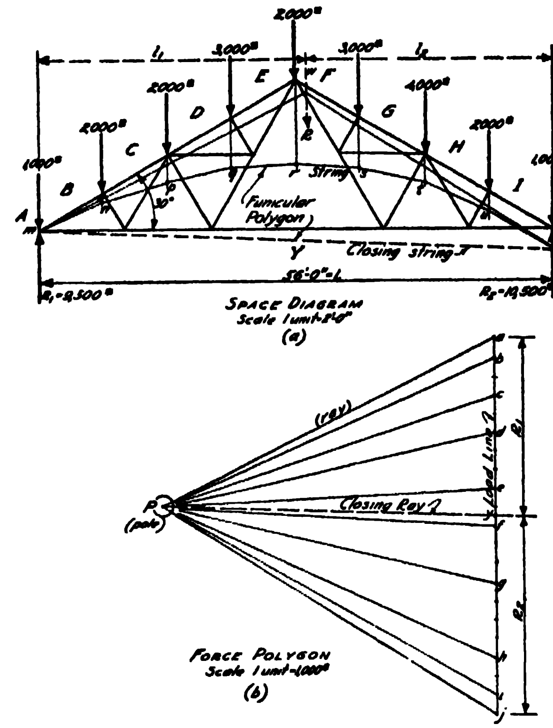


FIG. 286

drawn connecting the pole with the extremities of the loads, such as Pa, Pb, etc. These are called **rays.** The pole should be so selected that the length of the rays is sufficient to aid in transferring lines parallel to them, later in the solution. Experience will develop judgment in this respect as well as a gauge as to what will keep the solution, as carried along, within the limits of the paper.

The next step is to draw **strings** in the spaces between the loads in the space diagram, parallel to their respective rays. Thus the string mn in Fig. 286 (*a*) is drawn in space B parallel to the ray Pb in (*b*). Similarly, np is parallel to Pc; that in space D, parallel to Pd; the string in space E, parallel to Pe, and so on. Each string is extended until it cuts the line of action of the load terminating the space at either side.* The start may be made

* If a string parallel to the ray Pa were drawn starting from m, it must extend from this point until it cuts the line of action of the load terminating the space A. Since the point m is on the line of action of the latter, the string has no length and hence an attempt to draw a string parallel to Pa is not necessary. The same is true for Pj. If the strings were extended from m and v to intersect at w, the latter point would locate the line of action of the resultant of all the forces, R. R_1 and R_2 may be obtained by multiplying R by $l_2 + L$ and $l_1 + L$ respectively.

at any load but it is more natural to begin in the first space, namely B.* The first string may be drawn from any point on the line of action of the first load (in this case, 1000#), but it is more convenient to start at its point of application, m. Where the first string cuts the load between B and C, as at n, the second string is drawn to point p, and so on. The last string, namely uv, is drawn from u in space I, parallel to Pi, and cuts the line of action of R_2 at v. A line is now drawn to connect the starting and finishing points, such as mv. This is called the **closing string,** and it is generally indicated by a heavy dashed line. The whole figure, $mnpqrstuvm$ is called the **funicular polygon.**†

A line is then drawn from P in the force polygon parallel to the closing string mv cutting the load line at y. This then corresponds with the previous graphical work, as mv is in space Y, parallel to Py. Then jy represents R_2 and ya corresponds to R_1. These two forces are acting up, which corresponds with the reactions. In other words, if one goes from space J to space Y, R_2 intervenes and similarly from Y to A, R_1 occurs. The lengths jy and ya may now be scaled and their values obtained. If the diagram is followed around in a clockwise direction, it will be evident that it forms a closed polygon, which is a condition of equilibrium. When loads are applied to the bottom chord, the procedure is similar. The loads are plotted in order, going from one space to the next as before.

Prob. 186a. What is the value of each end reaction for the truss in Fig. 284 if the panel point loads are 4000# interior and 2000# at the ends?

Prob. 186b. Determine the values of R_1 and R_2 in Fig. 285 if the loads on the top chord are the same as shown except the peak load, which is to be 4000#, and for loads of 1000# applied at each bottom chord panel point. What is the effective end reaction in each case?

Prob. 186c. Verify the results in Prob. 186*a* by a graphical solution.

Prob. 186d. Solve Prob. 186*b* by a graphical solution.

187. Reactions Due to Inclined Loads.

The usual inclined loads affecting a truss are those caused by the action of the wind, since it is assumed to act normal to the roof surface. If inclined loads from some other source were to be considered, the reactions developed by them could be obtained in a manner similar to that for wind loads.

Since the loads are not vertical in such cases, a thrust is exerted upon the truss, which tends to overturn the supports. In other words, the reactions themselves are inclined. When vertical loads only are considered, as in the preceding article, it is of course known that the directions of the reactions are vertical, and all that is necessary is to establish their magnitudes. However, when the reactions are inclined, not only are the magnitudes of the reactions unknown, but their directions as well must be established. On such a basis, the problem is statically indeterminate, because only three basic laws of equilibrium, namely, $\Sigma H = 0$, $\Sigma V = 0$, and $\Sigma M = 0$, are available. Consequently it becomes necessary to make an assumption for a fourth condition.‡ This is done by qualifying the nature of the supports at the ends of the truss in either of two ways, namely:

(1) by assuming that both ends of the truss are fixed, or

(2) that one end of the truss is fixed, and the other is free to move in a specified direction.

It is of course necessary to choose which assumption shall be made for any given truss solution. The first assumes that the truss is rigidly anchored into masonry walls, and that these absorb the thrust or that a horizontal tie rod is to be used. This is the assumption commonly made for all trusses of short spans and for all wooden trusses. The second assumption is usually applied to steel trusses except in special instances. It may be divided into two cases, each qualifying the direction of the movement of the free end as:

2 (*a*) that the end not fixed is free to move only in a horizontal direction, and

2 (*b*) when the truss is supported by columns, that the movement at the free end depends upon the elastic deflection of the column.

The assumption 2 (*a*) is the more commonly adopted of these two. The free end is said to be on "rollers." Theoretically, the intention is that only horizontal motion is possible. If the rollers are perpendicular to the supporting surface and they are in good mechanical condition, they cannot offer resistance to motion along the surface. Therefore the reaction at the roller end is considered to be vertical, and the horizontal thrust of the wind must be provided for at the fixed end. Incidentally, rollers provide for expansion and contraction due to temperature changes. This form of support is supplied in practice by the use of sliding plates, cylindrical

* Care should be taken to cut the line of action of the load and not a web member which may be inclined.

† The location of the pole with respect to the load line will vary the position of the funicular polygon with respect to the truss diagram. That is, when P is to the left of the load line, the funicular polygon is above the bottom chord of the truss; when to the right, the funicular polygon is below the truss. As previously stated, the exact position of the pole does not affect the solution. It is interesting to use two poles and draw two diagrams to show that the values of the reactions in each case are the same.

‡ In all solutions, it is necessary to have as many available relations expressing conditions as there are unknowns.

rolls, or a rocker (see Index).* In the assumption 2 (*b*), the horizontal components of the wind load are assumed to be equal and to act at the theoretical points of inflection of the supporting columns. This case is common only with braced bent solutions and will be discussed in connection with them (see Index).

The solutions for the end reactions due to wind load will depend first upon what assumption is made relative to the fixity of the supports, as enumerated above. In dealing with the reactions, it is simpler to consider the total wind force as acting at one point, namely, the center of gravity of the wind panel loads, as illustrated in Fig. 287. Here W is the total wind force and acts at point n, on the center of gravity line of the wind panel loads.†

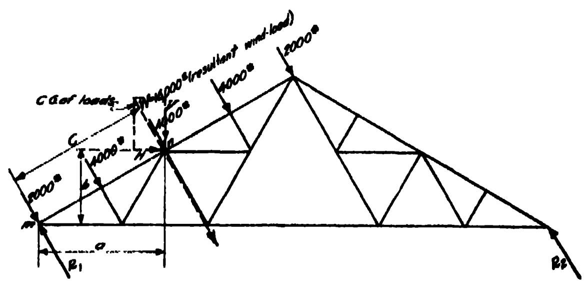


FIG. 287

In some cases it may be simpler to consider W' replaced by its horizontal and vertical components, H and V respectively. As in all instances, the values of the reactions may be obtained either analytically or graphically. The choice depends upon the particular solution, but in general the former process is preferable for ordinary trusses.

When **both ends** of the truss are assumed to be **fixed,** the truss is statically indeterminate. The horizontal component, H, of the total wind force, W, in Fig. 288 can be determined, but it is not known how much each support contributes to the resistance of this thrust. Consequently, it becomes necessary to make an additional assumption. This may be stated as

(1) the horizontal components of the two end reactions are equal, or

(2) the two reactions act in a direction parallel to the resultant wind load.

These two assumptions have no fixed relation to each other and the results obtained in each method will not agree exactly but are close enough for practical purposes. The assumption which is used is ordinarily dependent upon the kind of solution employed, the first for analytical calculations, and

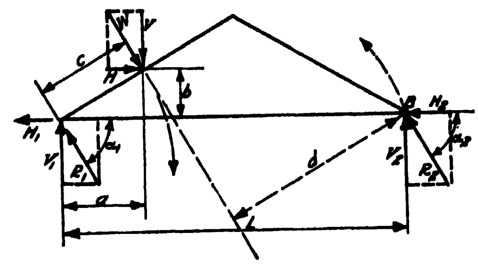


FIG. 288

the second for graphical constructions. In the **analytical solution,** the three basic laws of equilibrium, namely $\Sigma H = 0$, $\Sigma V = 0$, and $\Sigma M = 0$, may be used. Thus, in Fig. 288,

By $\Sigma H = 0$, $\quad H = H_1 + H_2$.

By assumption, $\quad H_1 = H_2$.

Therefore, $\quad H_1 = H_2 = \dfrac{H}{2}$.

By $\Sigma V = 0$, $\quad V = V_1 + V_2$.

By $\Sigma M = 0$, $\quad \Sigma M_A = W \cdot c - V_2 \cdot L = 0$, or

$$V_2 = \frac{W \cdot c}{L}, \text{ and}$$

$$V_1 = V - V_2.$$

$$\Sigma M_B = -W \cdot d + V_1 \cdot L, \text{ or}$$

$$V_1 = \frac{W \cdot d}{L}.$$

As an alternate solution for V_2,

$$\Sigma M_A = V \cdot a + H \cdot b - V_2 \cdot L = 0, \text{ or}$$

$$V_2 = \frac{V \cdot a + H \cdot b}{L}.$$

$$R_1 = \sqrt{V_1^2 + H_1^2} \text{ and } R_2 = \sqrt{V_2^2 + H_2^2}.$$

$$\tan \alpha_1 = \frac{V_1}{H_1} \text{ and } \tan \alpha_2 = \frac{V_2}{H_2}.$$

Thus it is seen that there are a number of relations, any of which may be used to obtain the desired values.

The corresponding **graphical solution** is similar to that discussed for obtaining vertical reactions. In Fig. 289, the spaces between the loads are assigned letters, and the loads, represented to scale, are drawn parallel to the direction of the wind. A pole is selected, the rays drawn, a funicular

* Other methods of support for large trusses are sometimes used, such as making the ends "hinged." In this case, the lines of action of the reactions are considered to act through the hinges. The latter are generally round pins of short length and a few inches in diameter, and rest in shoes. These in turn rest upon the supports. Hinges and rollers are occasionally combined. In this case the directions and the points of application of the reactions are both fixed. For large arched trusses, the rollers may be placed upon an inclined surface. In such cases, the reactions are considered to act in a direction perpendicular to the plane of the rollers. When tie rods are used to take the thrust, the lines of action of the reactions are automatically fixed (see Index).

† This is at the middle of the slant height for a group of symmetrical loads. When unsymmetrical or unequal loads are involved, the center of gravity must be located by calculations. If the wind load is normal to a roof composed of more than one slope the resultant direction of the wind load must also be obtained.

polygon drawn, and the closing line determined. Point y is determined by drawing Py parallel to mv. Then fy represents R_2 and ya, R_1. For the wind load on the right, the situation is just reversed, so that only one diagram is needed to obtain the maximum reactions.

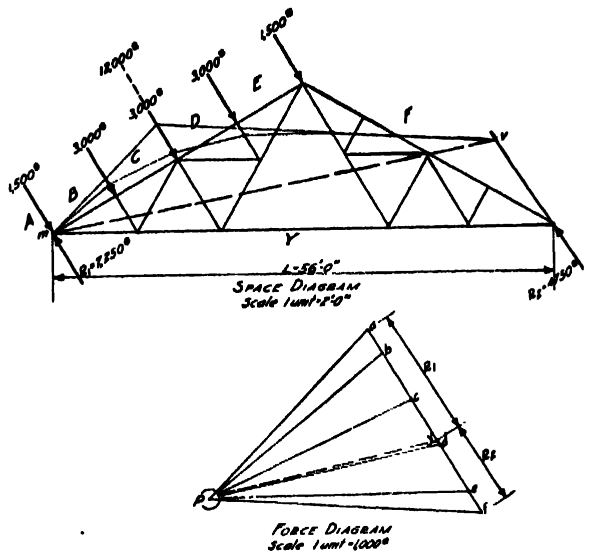


FIG. 289

When the assumption is made that only **one end** of the truss is **fixed** and that the **other is on rollers,** it is necessary to establish a governing condition. By common agreement **the rollers are always considered to be located at the right hand end of the truss.** This is done so that all designers will be in accord.* When their location is thus established, it is necessary to make two solutions, namely

(1) rollers at right, wind on left, and
(2) rollers at right, wind on right.

A study of these two cases will reveal that each develops different stresses in the members. Both cases must be investigated in order to determine the maximum stress in any given member. **Analytical solutions** for the reactions are similar to those previously discussed. Referring to Fig. 290 (a), the three basic laws of equilibrium are applied to the case of the wind load on the left, as follows:

By $\Sigma H = 0$, $H - H_1 = 0$, or $H_1 = H$.
By $\Sigma M = 0$, $\Sigma M_A = W \cdot c - R_2 \cdot L$, or

$$R_2 = \frac{W \cdot c}{L}.$$

By $\Sigma V = 0$, $V = V_1 + R_2$, or

$$V_1 = V - R_2$$
$$R_1 = \sqrt{H_1^2 + V_1^2}$$
$$\tan \alpha_1 = \frac{V_1}{H_1}.$$

* The final results will, however, be the same.

When the wind is acting from the right, as in Fig. 290 (b), the reactions are obtained as follows:

By $\Sigma H = 0$, $-H + H_1 = 0$, or $H_1 = H$.
By $\Sigma M = 0$, $\Sigma M_A = W \cdot e - R_2 \cdot L = 0$, or

$$R_2 = \frac{W \cdot e}{L}.$$

By $\Sigma V = 0$; $V = V_1 + R_2$, or $V_1 = V - R_2$

$$R_1 = \sqrt{H_1^2 + V_1^2}$$
$$\tan \alpha_1 = \frac{V_1}{H_1}.$$

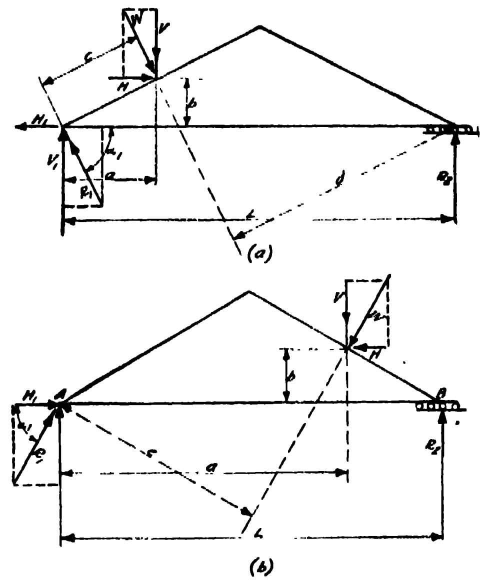


FIG. 290

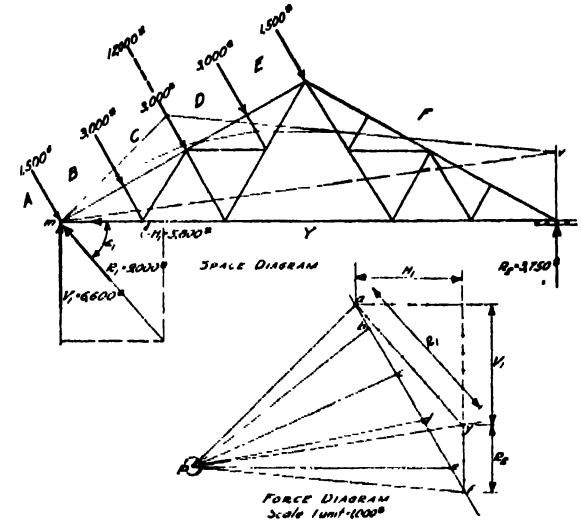


FIG. 291

The corresponding **graphical solutions** are similar to those discussed before. That in Fig. 291 is for the wind load acting on the left. The spaces are lettered, the load line constructed, and the funicular

polygon drawn as in previous work. Py is drawn parallel to the closing line mv to cut a vertical from f, as R_2 is specified to act vertically. The values of fy and ya represent R_2 and R_1 respectively. The horizontal components of the latter may now be obtained. Figure 292 shows the solution of the reactions for the same truss and specifications, with the wind acting on the right.

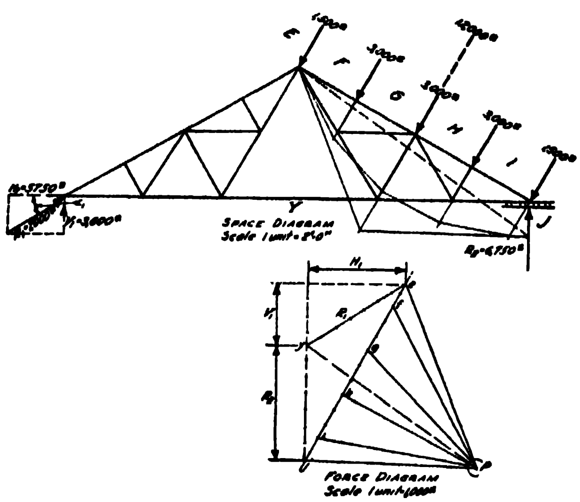


FIG. 292

Prob. 187a. Calculate the values of R_1 and R_2 in Fig. 289 if the truss is assumed with fixed ends and the pitch is 30°.

Prob. 187b. Determine the values of R_1 and R_2 in Fig. 287 graphically, if the truss is ¼ pitch and the span 60′-0″, fixed ends.

Prob. 187c. Repeat Prob. 187*b* if the right hand end of the truss is on rollers (graphical solution).

Prob. 187d. Reverse the wind load to the other side of the truss of Probs. 187*b* and 187*c* and determine R_1 and R_2 (graphical solution).

Prob. 187e. Obtain R_1 and R_2 in Prob. 187*c* by an analytical solution.

Prob. 187f. Obtain R_1 and R_2 in Prob. 187*d* by an analytical solution.

188. Methods of Determining Stresses.

When the reactions caused by the different conditions of loading are established, the next problem in the design of a truss is to determine the stresses in the members, due to the external forces, namely, the loads and their reactions. The stresses due to each kind of loading are generally computed separately and later combined to determine their maximum combined effect. The same general principles apply to each condition of loading.

The natures of the stresses can usually be determined by inspection. Thus in Fig. 293 (*a*), the loads tend to bend the truss in a curve convex downward, similar to the action of a simple beam. Hence in simple trusses, the top chord members are in compression and the bottom chord members are in tension. The shearing force, V, must be offset by a downward force, C. Since the directions of stress are always opposed at the ends of a member, the reaction of C must act toward b. To balance this, the stress in bc must be tension, and so on. Therefore the stresses in the web members, in this case, are alternately compression and tension. The end inclined members of parallel chord trusses are in compression. When a member frames into another at right angles, as mn into pr in Fig. 293 (*b*), and no load is applied at n, the stress in mn is theoretically zero.

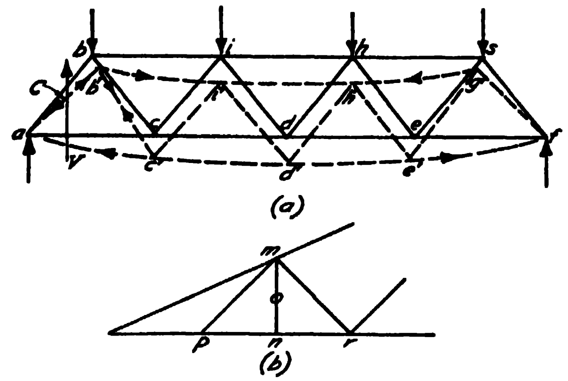


FIG. 293

The magnitude of the stresses in truss members may be found in a number of ways, either involving analytical or graphical solutions. These will be discussed in the following text, as it is desirable for a designer to be able to determine stresses either way, as one method may be made to serve as a check upon another, and in many cases, one may be simpler to apply than another. In general, the graphical method is more commonly used and is preferable for trusses with non-parallel chords. The disadvantage which is evident is that diagrams must be laid out to a scale, whereas the analytical solution does not require any drafting. The accuracy is naturally limited by the delineation of the graphical figures, but it is sufficient when ordinary care is used, particularly when the accuracy of the loads causing the stresses is limited by usual load assumptions. The algebraic methods are often too laborious, but they offer the advantage that a particular stress may be solved directly for a check upon the graphical solution. They are easily adaptable to trusses with parallel chords.

As stated above, there are a number of so-called methods in algebraic solutions, which are more or less interrelated. These may be enumerated as:

(1) the method of joints,
(2) the method of moments,
(3) the method of shears, and
(4) the use of coefficient tables.

When each of the above is understood, all may be more or less combined in any analytical solution, as one method may be more rapid than another, for the determination of a particular stress. In all analytical solutions, it is usually advisable to first establish the distances between all the truss joints.

In all truss solutions, it is necessary to make certain assumptions. These are commonly grouped as follows:

(1) the frame is not confined by the reactions,

(2) the axes of the members all meet in common points, and

(3) the joints are assumed to act as if they were frictionless hinges.

189. Graphical Solutions.

In a graphical solution to determine the stresses in a truss, it is first necessary to construct a diagram of the frame at a convenient scale and to find the values of the end reactions either analytically or graphically. If these values were obtained algebraically, the corresponding points on the load line, previously laid out at a convenient scale, should be located on it.

The solution for the stresses may be made by analyzing each joint separately.* Each is considered to be a free body and the simple applications to concurrent forces in equilibrium may be used, namely, that the forces drawn parallel to their lines of action must form a closed polygon. Referring to Fig. 294, the joint at the top of the A-truss, when taken out as a free body, must be in equilibrium.† The load P is the cause of the stresses in A–1 and B–1 and hence is the "resultant," or the diagonal of a parallelogram formed on it, with sides parallel to the two members.

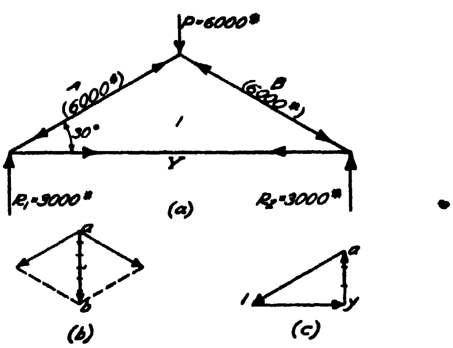


FIG. 294

This is illustrated in (b) and is simply the principle of the resolution of forces. Similarly, the reaction at the left may be considered the resultant of the stresses A–1 and Y–1. Figure 294 (c) illustrates the triangle of forces drawn. With ab and ya plotted to a convenient scale and the forces parallel to their respective members, the lengths $a1$ and $y1$ may be scaled to establish the compression in the top chord and the tension in the bottom chord, respectively.

The same procedure may be applied to a larger truss as in Fig. 295. If the joint at the left hand end of the truss is taken out as a free body, as in (b), a triangle of forces may be completed with 7500# = by, a vertical force, and the other two sides parallel to the top and bottom chords. The sides b–1 and y–1 may now be scaled to establish the stresses in the respective members. It should be noted that a–b (=1500# in this case) does not affect the stresses because it has the same line of action as the reaction, yet it is opposite in direction. Hence it may be subtracted from the total end reaction, and the effective reaction is 9000# − 1500# = 7500#. The effective forces at any joint, when plotted in clockwise order, must make a closed polygon to satisfy the imposed condition of equilibrium. Applying the reverse of this to maintain equilibrium, when some of the forces at a joint are known in magnitude and direction, the values of the others may be found, since their directions are known, by completing the polygon with the closing sides parallel to the unknown forces. This means that the unknown stresses at any joint must not exceed two, as it would not then be possible to complete the polygon. Consequently the order in which the joints may be analyzed is controlled by this requirement. Referring to Fig. 295 again, the panel point at load B–C may be investigated, as the stress in 1–2 is 0. The latter is true because the member frames into Y–1 at right angles and there is no applied load there. With 1–2 = 0, there are only two unknowns left, and the stresses C–3 and 3–2 may be obtained as in (c). The other joints may then be treated in a similar manner, as illustrated in Fig. 295 (d) to (g) inclusive.

Separate solutions at each joint do not have to be made graphically, and the usual procedure is to combine them into one diagram, as it is simpler. One advantage is that mistakes will become manifest when the polygon does not make a closed figure. The first step in a solution is to lay out the load line, including the reactions, as previously discussed. This is illustrated in Fig. 296 for the same truss previously considered for the separate joints. The method of denoting the loads, both on the truss and in the force diagram, is called to attention again (Art. 186). The spaces between the web members

* This is simply the graphical method corresponding to the analytical method of joints (Art. 190).

† This is a basic law of structural mechanics. Any structure as a whole when taken out as a free body must be in equilibrium. Similarly, any member of the structure, or any part of any member when treated in a like manner, must also be in equilibrium.

are assigned arabic numerals and these are in consecutive order from left to right. The points corresponding in each diagram should be studied in order to note their relations. This is often called **Bow's notation.** Several advantages result from the system shown. Thus all top chord members are recognized by a first letter of the alphabet and a numeral, such as *B*–1, *C*–3, etc. All bottom chord members are recognized by the letter *Y*, followed by a number, such as *Y*–1, *Y*–3, etc. All web members may be identified by numerals, as 1–2, 2–3, and so on.

When the loads and the reactions are completely represented, the stresses may be obtained. Thus in Fig. 296, the load *B*–*C* is known in amount and direction, and the direction of the stress in *B*–1 is known. The latter is always established, since the natural path of a stress is parallel to the axis of a member. Accordingly, a line of indefinite length may be drawn from *b* in the force diagram, parallel to *B*–1 in the space diagram. Similarly, a line of indefinite length through *y* may be drawn parallel to *Y*–1. The point 1 is located by their intersection. The length of *b*–1, measured to the same scale as the load line, represents the stress in *B*–1; and *y*–1, the stress in *Y*–1. Point 2 is located by drawing lines from 1 and *c* parallel to 1–2 and *C*–2 respectively. Following the same procedure, the remaining points, and subsequently the corresponding stresses, may be established, as illustrated in the figure.

It is not only necessary to obtain the amount of stress in each truss member, but also its kind, namely, tension or compression. When a member is subjected to tensile forces, the reaction, or stress set up in the member, opposes the external forces. Thus in Fig. 296, the internal forces are represented as acting toward each other, or pulling on the end joints. Similarly in compression, the arrows are pointed away from each other. The kind of stress may be obtained from the force diagram. This is done by tracing the forces around any joint, starting in any space and proceeding always in a clock-

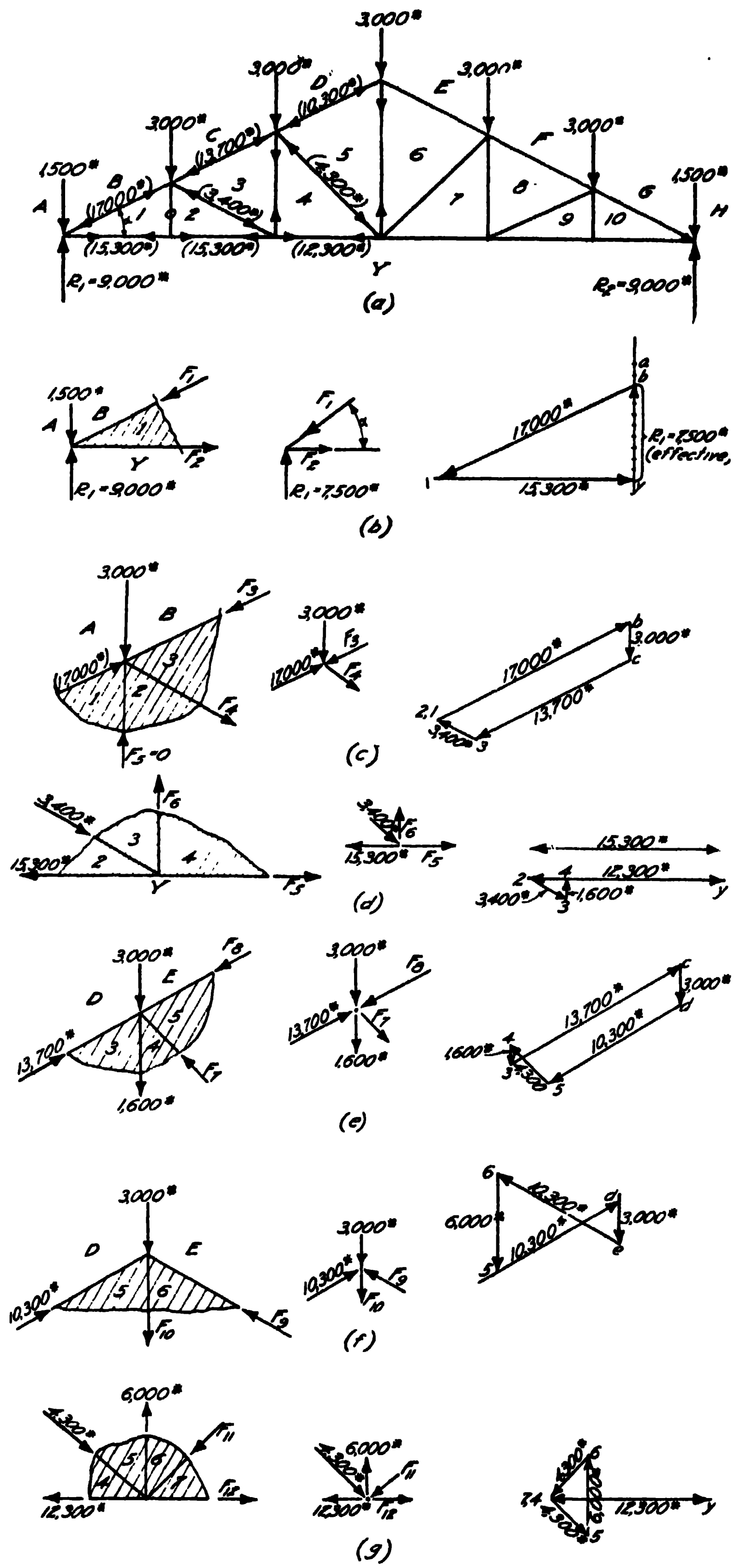


FIG. 295

wise direction with the corresponding forces in the truss diagram. This may be illustrated by a reference to Fig. 296 for the first top chord joint, namely, at the load B–C. In the force diagram, b–c is vertically down, which corresponds with the load. From c to 3 in the force diagram is diagonally down and to the left. Moving in a clockwise di-

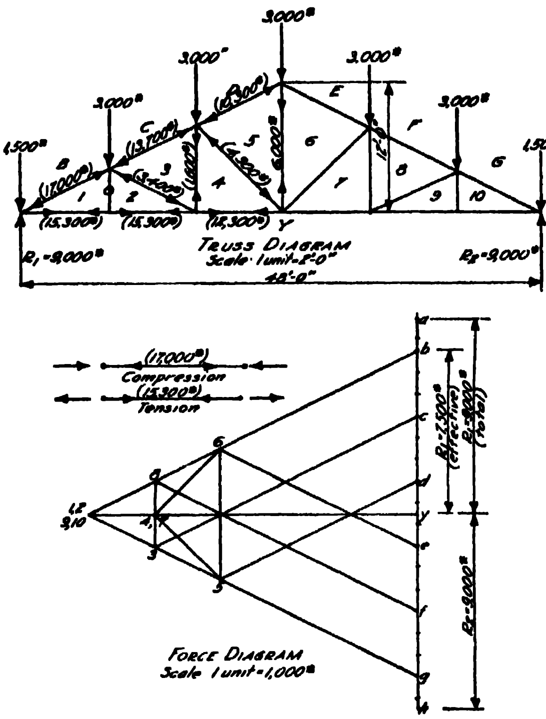


Fig. 296

rection from space C to space 3 in the truss diagram, it is seen that the stress in the member must correspond. It pushes toward the joint, and therefore C–2 is in compression. Likewise when moving from space 3 to space 2 in the truss diagram, and noting the direction of 3–2 in the force polygon to be up, the stress in member 3–2 is compression. The distance from 2 to 1 is zero so that the stress is theoretically zero. Similarly the member 1–B is in compression. The kind of stress in any other member may be obtained by the same procedure. The stresses should always be indicated on the member, as shown on the truss diagram, and not on the force diagram. In this way the stresses are placed where they really act and the action of the truss can be more easily visualized. A separate table of stresses for each individual case beside the diagram is of no particular value (see Art. 194). When the loads are equal and symmetrically arranged with respect to the span of the truss, only a half-force diagram is necessary, as the stresses are naturally the same in the members corresponding on either side.

The solutions for other kinds of loading are made in the same manner as has been demonstrated, once the load line and the corresponding reactions have been established (Art. 186). No diagram need be drawn for the stresses due to snow load if those due to dead load have been obtained. The reason for this is that snow loads are considered to act vertically. The stresses may then be obtained by direct proportion of the panel loads. Figures 297, 298, and 299 show several types of graphical solutions for various kinds of trusses and loadings.

Prob. 189a. Determine the stresses in an A truss (similar to Fig. 294) for a peak load of 4000#, a span of 22′-0″ and a ¼ pitch roof (graphical solution).

Prob. 189b. Determine the stresses in Fig. 296 for panel point loads of 2500# each (graphical solution).

Prob. 189c. Repeat Prob. 189b for an eight panel truss with span of 60′-0″, ¼ pitch.

190. Method of Joints (Analytical).

As previously stated, any joint of a truss which is taken out as a free body must be in equilibrium. This may be done by using an imaginary section line, as illustrated in Fig. 300 (b). Since all the members intersect in a common point at every joint, in accordance with the assumption made in the general truss theory, the problems are those of concurrent forces. Such forces do not cause any moment about their point of intersection. This eliminates one of the laws of equilibrium, namely, $\Sigma M = 0$, and only the other two are available, — that is, $\Sigma H = 0$ and $\Sigma V = 0$. Therefore the section line selected should not cut the lines of action of more than two unknown forces. The reason for this is that only two conditional equations can be stated. With such limitations, there are only certain joints in a truss where a solution may be completed. This usually means that it is necessary to start at the support of a truss.

The above discussion may be illustrated by a reference to Fig. 300. The two members cut by the section line a–a are replaced by external forces acting in the direction of the member. These are necessary to produce a condition of equilibrium as in (b). If the directions of these forces are not evident at first, they may be assumed. The algebraic sign of the force as it is determined will indicate whether the direction was assumed correctly or not. If the value resulting has a positive sign, the direction assumed is correct; if it has a negative sign, the direction of that force should thereafter be considered as opposite to that first assumed. The correct directions indicate the kind of stress in the member, remembering that a force toward the joint indicates compression in the member, and one away from the joint, tension. The best procedure is to select a pair of horizontal and vertical axes

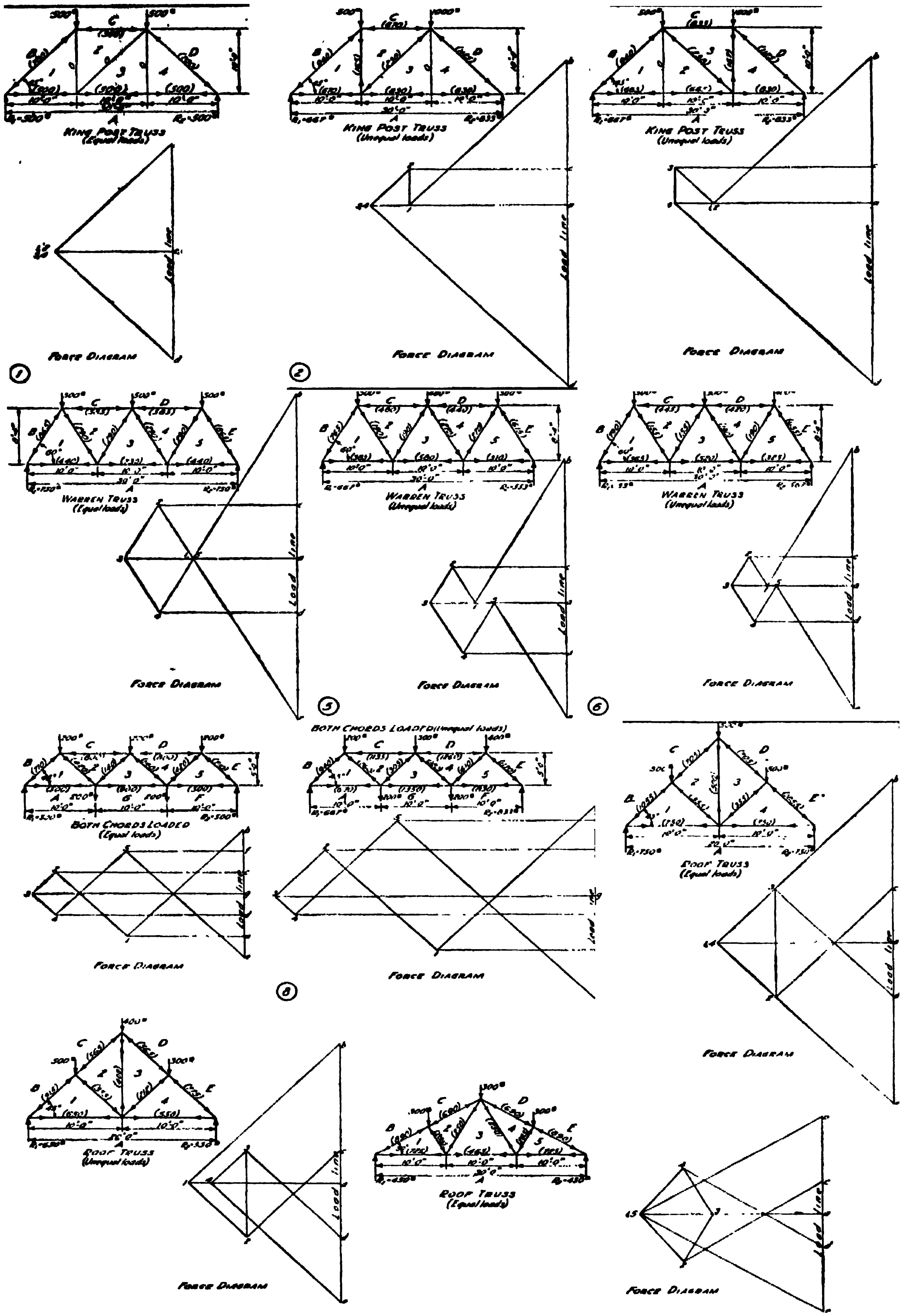

King Post Truss
(Equal loads)
Force Diagram
King Post Truss
(Unequal loads)
Force Diagram
King Post Truss
(Unequal loads)
Force Diagram
Warren Truss
(Equal loads)
Force Diagram
Warren Truss
(Unequal loads)
Force Diagram
Warren Truss
(Unequal loads)
Force Diagram
Both Chords Loaded (Unequal loads)
Both Chords Loaded
(Equal loads)
Force Diagram
Force Diagram
Roof Truss
(Equal loads)
Force Diagram
Roof Truss
(Unequal loads)
Force Diagram
Roof Truss
(Equal loads)
Force Diagram

Fig. 297

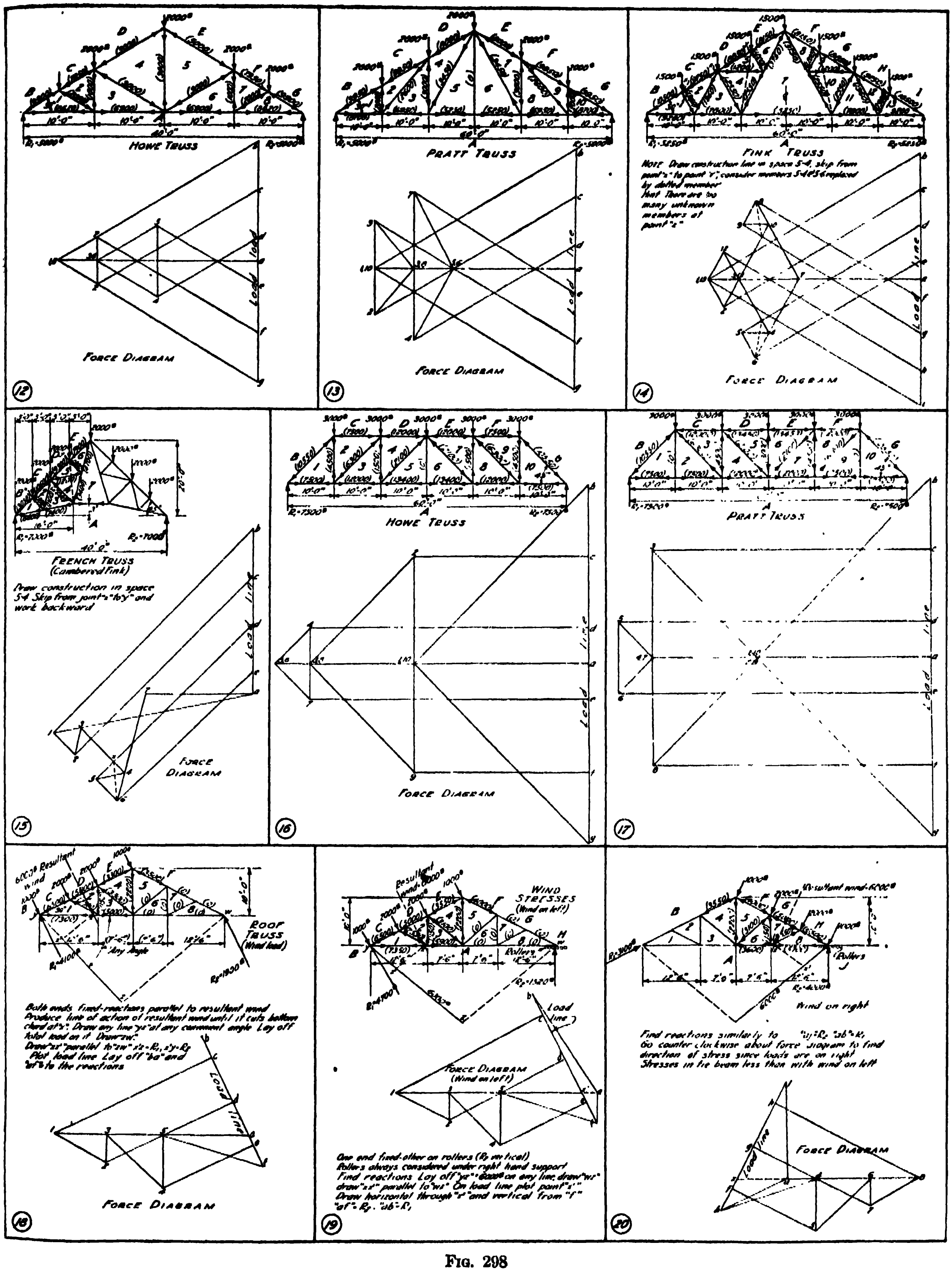

HOWE TRUSS
FORCE DIAGRAM
PRATT TRUSS
FORCE DIAGRAM
FINK TRUSS
NOTE Draw construction line in space 5-4, skip from point "z" to point "r", consider members 5-4 & 5-6 replaced by dotted member
that There are too many unknown members at point "z"
FORCE DIAGRAM
FRENCH TRUSS (Cambered Fink)
Draw construction in space 5-4 Skip from joint "z" to "y" and work backward
FORCE DIAGRAM
HOWE TRUSS
FORCE DIAGRAM
PRATT TRUSS
ROOF TRUSS (Wind load)
Any Angle
Both ends fixed-reactions parallel to resultant wind
Produce line of action of resultant wind until it cuts bottom chord at "x". Draw any line "ys" at any convenient angle. Lay off total load on it. Draw "zw".
Draw "xs" parallel to "zw" x'z = R₁, x'y = R₂
Plot load line Lay off "ba" and "af" to the reactions
FORCE DIAGRAM
WIND STRESSES (Wind on left)
Rollers
FORCE DIAGRAM (Wind on left)
One end fixed-other on rollers (R₂ vertical)
Rollers always considered under right hand support
Find reactions Lay off "ys" = 6000# on any line, draw "ws"
draw "xt" parallel to "ws" On load line plot point "t"
Draw horizontal through "t" and vertical from "f"
"af" = R₂. "ab" = R₁
Rollers
Wind on right
Find reactions similarly to "aj" = R₂ "ab" = R₁
Go counter clockwise about force diagram to find direction of stress since loads are on right
Stresses in tie beam less than with wind on left
FORCE DIAGRAM

Fig. 298

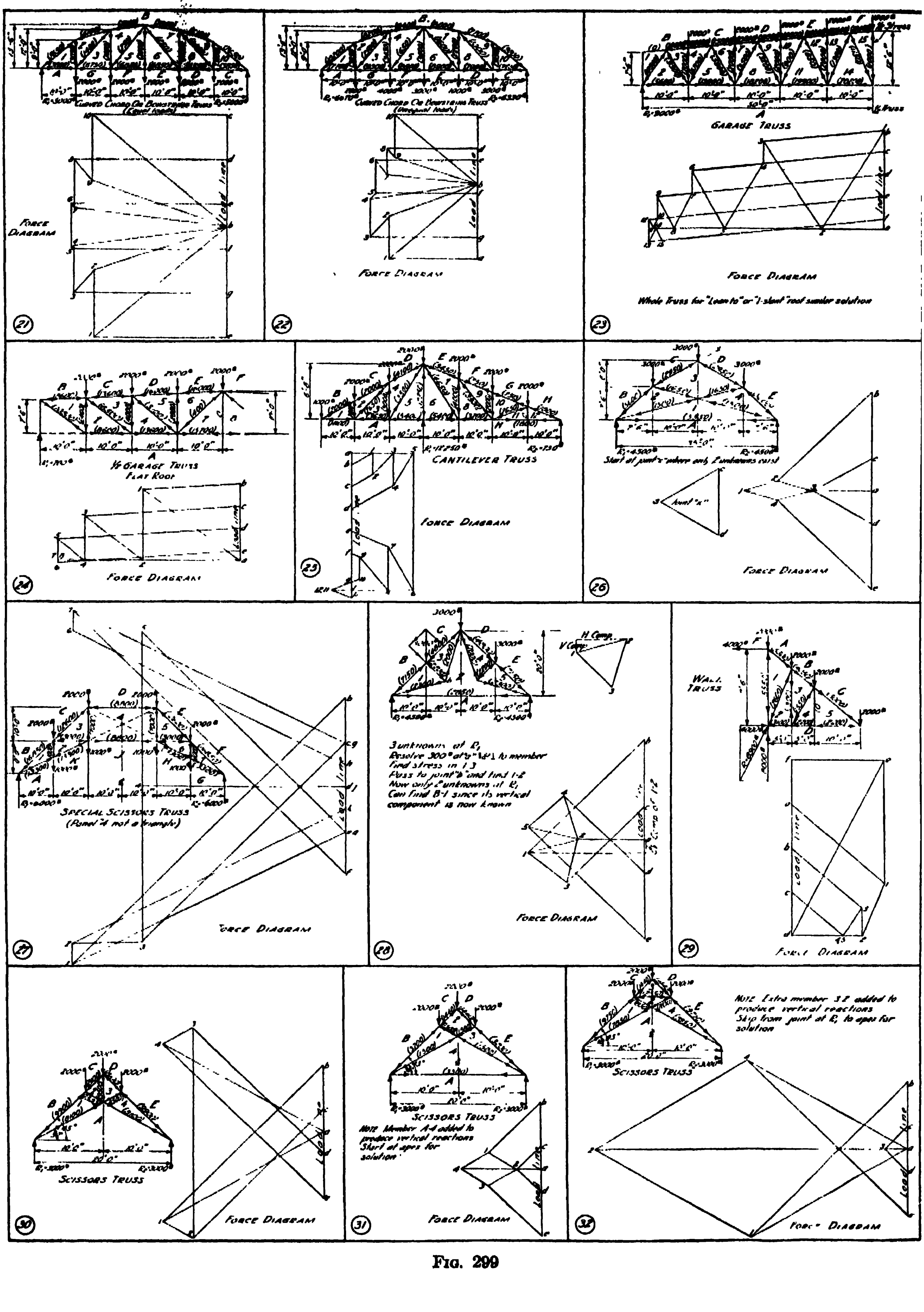

Force Diagram
Garage Truss
Force Diagram
Whole Truss for "Lean to" or "1-slant" roof similar solution
½ Garage Truss
Flat Roof
Force Diagram
Cantilever Truss
Force Diagram
Force Diagram
Special Scissors Truss
(Panel "4 not a triangle)
Force Diagram
Force Diagram
Wall Truss
Force Diagram
Scissors Truss
Force Diagram
Scissors Truss
Force Diagram
Scissors Truss
Force Diagram

FIG. 299

through the joint, and to take a summation of horizontal and vertical components. The axes should be so selected as to be coincident with the greatest number of members possible. The joint may be rotated as a whole to accomplish this if desired. A little practice will develop ability to accomplish these two things. Referring again to Fig. 300 (*b*), the following equations and the corresponding stresses result:

By $\Sigma V = 0$, $R_1 - F_1 \cdot \sin\theta = 0$, or
$4500 - F_1 \cdot \sin 30° (= 0.5) = 0$
$F_1 = 9000\#$.

The sign is +. Hence the direction was assumed correctly. Therefore the stress in A–1 = 9000# compression.

By $\Sigma H = 0$, $F_2 - F_1 \cdot \cos 30° = 0$,
or $F_2 - 9000 \times 0.866 = 0$
$F_2 = 7800\# =$ stress Y–1, tension.

When two such stresses have been determined, another section involving two unknown forces (not more than two) may be considered as a free body, as in Fig. 300 (*c*) and (*d*). That in (*d*) is the joint in (*c*) revolved through 30° for convenience. The joint at Y–1–2–3 cannot be considered at this time since there are three unknowns, nor the one at the apex of the truss. In Fig. 300 (*d*), the following results:

By $\Sigma V = 0$, $-F_4 + 3000 \times \sin 60° = 0$
$-F_4 + 3000 \times 0.866 = 0$
$F_4 = 2600\# =$ stress 1–2, compression.

In an analysis of the forces in general, F_1 and F_3 cannot resist the vertical effect of the 3000# force, and F_4 is the only member left to supply this resistance. Similarly, F_4 contributes no horizontal resistance, or

By $\Sigma H = 0$, $-F_1 + F_3 + 3000 \times \cos 60° = 0$
$-9000 + F_3 + 3000 \times 0.5 = 0$
$F_3 = 7500\# =$ stress B–2, compression.

A little experience will develop intuition to select sections which involve only two unknowns. The joint shown in (*e*) may now be analyzed as follows:

By $\Sigma V = 0$, $-F_4 \cdot \sin 60° + F_5 \cdot \sin 60° = 0$
$-2600 \times 0.866 + F_5 \times 0.866 = 0$
$F_5 = 2600\# =$ stress 2–3, tension.

By $\Sigma H = 0$, $-F_2 + F_4 \cdot \cos 60° + F_5 \cdot \cos 60° + F_6 = 0$
$-7800\# + 2600 \times 0.5 + 2600 \times 0.5 + F_6 = 0$
$F_6 = 5200\# =$ stress Y–3, tension.

The joint at the peak of the truss may be taken out as a free body to check the solution if desired. A disadvantage of this type of solution is that when

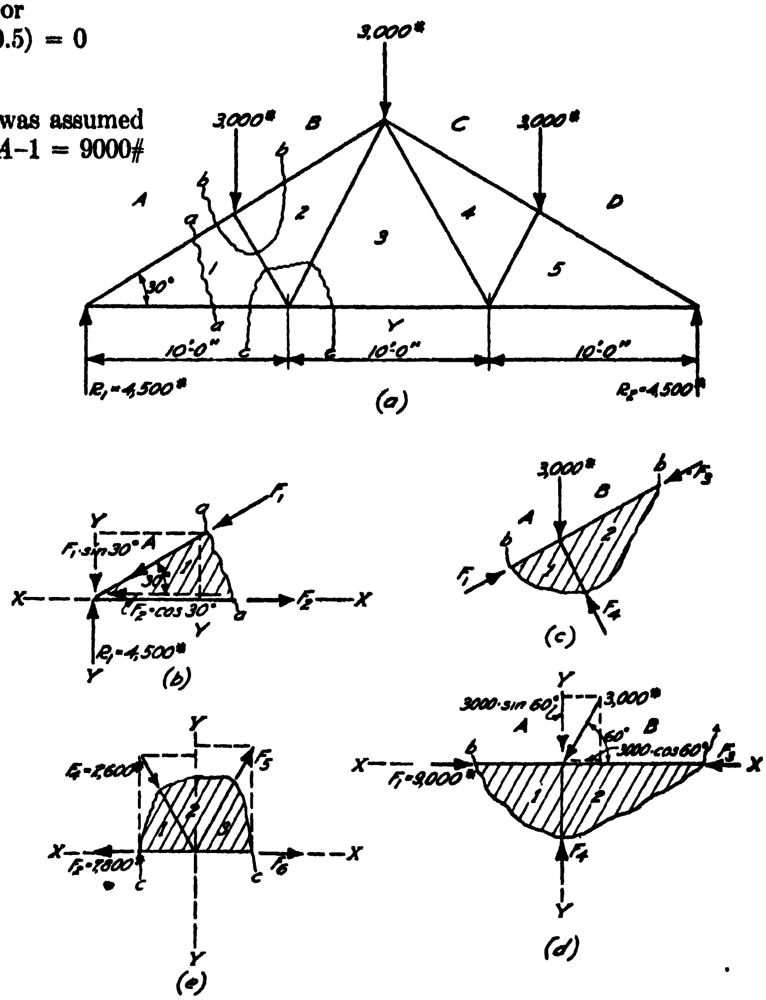


FIG. 300

any particular stress is wanted, all the joints from one end of the truss up to the one involving that member must be analyzed first. The stresses, as they are obtained, should be indicated both in amount and kind upon the truss diagram.*

Prob. 190a. Determine the stresses in the members of the truss shown in Fig. 296 (previously discussed in Art. 189) by an analytical solution.

* With experience it is not necessary to actually draw each section as a separate figure, and these may be analysed by imagining the sections, or by drawing light section lines on the truss diagram only.

$\tan \alpha = \frac{12}{24} = 0.5 \quad \alpha = 26° 34'$

$\cos \alpha = 0.894 \quad \sin \alpha = 0.447.$

Effective end reaction $R_1 = 9000 - 1500 = 7500\#$.
Neglect 1500# at end in stress solution.

Prob. 190b. Determine the stresses in Fig. 294 by analytical methods.

Prob. 190c. Determine the stresses in Fig. 297 (5) by the method of joints.

Prob. 190d. What are the values of the stresses as determined in Fig. 297 (11) by analytical methods?

191. Method of Moments (Sections).*

Another analytical method which may be used to determine the stresses in truss members is that of moments. As before, sections may be used. Since the truss as a whole is in equilibrium, any part of it taken as a free body must also be. The truss may be considered as cut by any imaginary section line into two parts, one of which may be considered for stress analysis. The members cut by the imaginary section line are considered to be acted upon by equivalent forces as in the method of joints. The portion of the truss under consideration is in equilibrium by the action of the known external forces and the unknown forces representing the stresses. The directions and the kinds of stress are determined, as previously explained. A point of difference in the two methods is that in this procedure the members cut need not all intersect in a common point. This being true, one or more forces tend to rotate about any given point. The moments of all the forces about such a point must be equal to zero if equilibrium is to exist. Consequently all three laws of equilibrium are available, namely $\Sigma H = 0$, $\Sigma V = 0$, and $\Sigma M = 0$. The section line drawn should not cut more than three members in which the stresses are unknown, since there are only three equations which may be used. If more than three are cut, the section under consideration is indeterminate.

The equation for $\Sigma M = 0$ may be stated with reference to any point in the plane of the truss members, but it is naturally more convenient to select a point which has a definite relation to the members. If the moment center is taken at the intersection of the lines of action of two of the unknown stresses, the latter have no moment about that point. On this basis, $\Sigma M = 0$ is the only law which need be applied and the stress in the member not passing through the point may be solved for directly. A little practice of this kind will soon develop ability to select proper moment centers. The method applies most favorably when the top and bottom chords are horizontal, although it may be applied to all trusses.

* Often called Ritter's method, originally developed by Rankine.

Prob. 191a. Determine the stresses in the members of the truss shown in Fig. 301 by the method of moments.

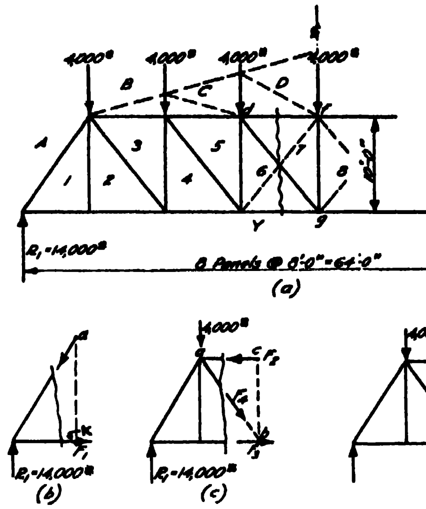


FIG. 301

192. Method of Shears (For Web Members).

A third analytical method which may be used to determine the stresses in the web members of trusses, particularly those with parallel chords, is that called the method of shears. It is used the least of any of the three so far discussed, especially for building work, as it is only an auxiliary to other methods.† If a truss with horizontal top and bottom chords has only one inclined member in a given panel, the latter is the only member which can offer a resistance to the shear in the panel since the chords are horizontal. In other words, the shear in the panel is the vertical component of the stress in the inclined member.

Referring to Fig. 302 (a), the shear at plane x–x is 15,000#. This is also the vertical component of the stress in member A–1. This stress is then $15{,}000 \div \sin 45° = 21{,}200\#$. Similarly, the shear at plane a–a is $15{,}000 - 5000 = 10{,}000\#$, and the stress in 1–2 is $10{,}000 \div \sin 45° = 14{,}100\#$. The kind of stress may be determined from the direction of the shear. Thus in Fig. 302 (b), V_L ($= 10{,}000\#$) acts up, and the force F_1 must pull down to oppose it. Hence the stress in member 1–2 is tension. It should be obvious that Y–1 and B–2 can contribute no resistance toward vertical motion. If the right hand portion in (b) is considered, the shear at a–a, V_R, is opposite to V_L, and F_1 must act up. This corroborates the previous analogy. Similarly, the shear at b–b is 10,000# up and the force F_2, or member 1–2 in Fig. 302 (a), must push down. The stress in 1–2 is therefore compression. The general rule that the stresses in the web members are alternately compression

† This is very common in bridge truss design, however.

and tension aids in determining the kind of stress in any given member, also. In this method the chord stresses are usually determined by the method of moments (Art. 191). For trusses with horizontal top and bottom chords, the stresses in these members are quickly found with the knowledge that the stress

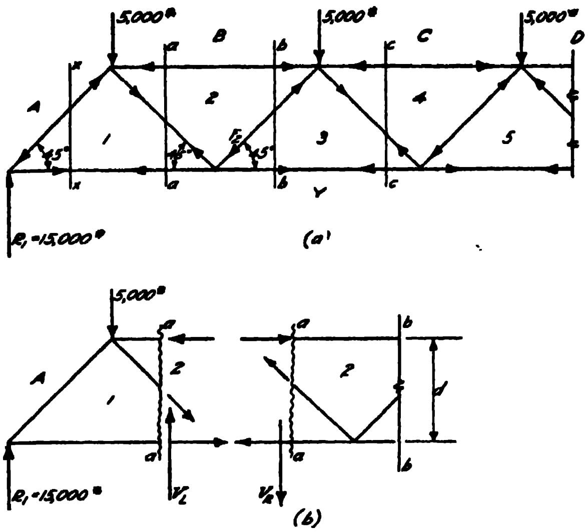


FIG. 302

is equal to the moment in the panel divided by the depth of the truss. This is similar to obtaining the tension or compression in any simple beam, namely, by dividing the moment at a given point by the effective depth of the beam.

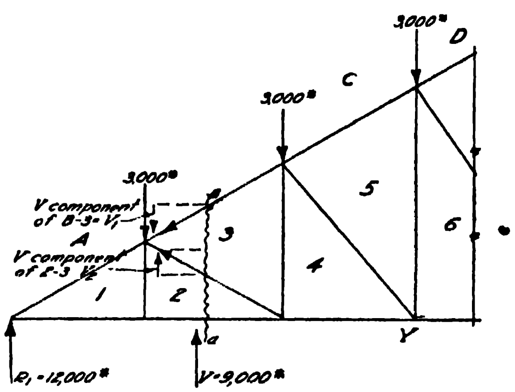


FIG. 303

The method of shears may be indirectly used in determining the stresses in the web members of trusses with an inclined top chord, if the chord stresses have already been established. Thus in Fig. 303, when the stress in B–3 is known, its vertical component may be obtained. The algebraic sum of this and the vertical component of the stress in 2–3 must balance the shear in the panel. Hence $+V(=9000\#) - 3000\# - V_1 + V_2 = 0$ in the figure. The reverse may also be used. That is, the stress in B–3 may be found if that in 2–3 has been determined by another method.

Prob. 192a. Determine the stresses in the web members of the truss shown in Fig. 301 by the method of shears.

Prob. 192b. Calculate the stresses in the web members of the truss in Fig. 304 (*a*) by the method of shears.

Prob. 192c. Calculate the stresses in all the members of the truss shown in Fig. 304 (*b*). Use any combination of joints, moments and shears desired.

Prob. 192d. Calculate the stresses in the members of the truss shown in Fig. 304 (*c*) by any method.

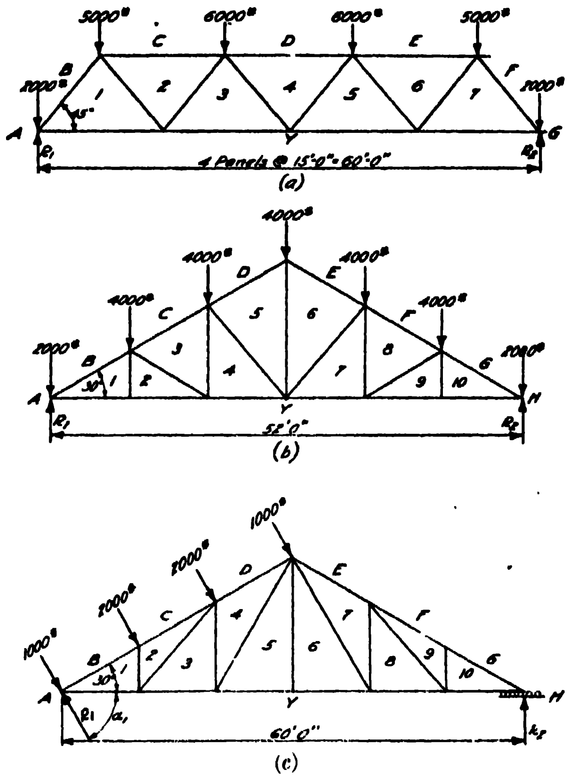


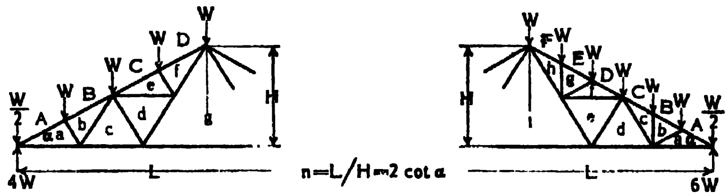
FIG. 304

193. Use of Coefficient Tables.

Another analytical solution for the stresses in trusses may be made with the aid of tables of coefficients. These are a number of tabulated factors referring to a given type of truss and which are expressed as a function of the form of the framing and of the relations of the loads. They are usually given in terms of the panel loads and the distances are given in terms of the rise and the span of the truss. The stress in any member is obtained by multiplying the panel load by its corresponding coefficient. Such tables as these are given in a number of books and they will not be repeated here.* Figure 305 illustrates the general nature of these tables.

Where there are a number of trusses of a common type, and the shape, pitch, and loading correspond to a set of coefficients, such tables are often a distinct advantage. This occurs usually in the case where the panel loads are all equal. However,

* Refer to the "Pocket Companion," Carnegie Steel Co., or to Hool and Johnson's "Building Construction," Vol. I, — copyright McGraw-Hill Book Co., Inc.



COMPOUND FINK TRUSS

Member	Stress		Length
Aa	$+\frac{7}{4}\sqrt{n^2+4}$	$\times W$	$\frac{1}{8}$ L sec α
Bb	$+\frac{1}{\sqrt{n^2+4}}(\frac{7}{4}n^2+5)$	$\times W$	$\frac{1}{8}$ L sec α
Ce	$+\frac{1}{\sqrt{n^2+4}}(\frac{7}{4}n^2+3)$	$\times W$	$\frac{1}{8}$ L sec α
Df	$+\frac{1}{\sqrt{n^2+4}}(\frac{7}{4}n^2+1)$	$\times W$	$\frac{1}{8}$ L sec α
La	$-\frac{7}{4}n$	$\times W$	$\frac{1}{8}$ L $\sec^2\alpha$
Lc	$-\frac{3}{2}n$	$\times W$	$\frac{1}{8}$ L $\sec^2\alpha$
Lg	$-n$	$\times W$	L $(1-\frac{1}{2}\sec^2\alpha)$
ab, ef	$+\frac{n}{\sqrt{n^2+4}}$	$\times W$	$\frac{1}{8}$ L sec α tan α
cd	$+\frac{2n}{\sqrt{n^2+4}}$	$\times W$	$\frac{1}{4}$ L sec α tan α
bc, de	$-\frac{1}{4}n$	$\times W$	$\frac{1}{8}$ L $\sec^2\alpha$
dg	$-\frac{1}{2}n$	$\times W$	$\frac{1}{4}$ L $\sec^2\alpha$
fg	$-\frac{3}{4}n$	$\times W$	$\frac{3}{8}$ L $\sec^2\alpha$

COMPOUND FAN TRUSS

Member	Stress		Length
Aa	$+\frac{1}{\sqrt{n^2+4}}(\frac{11}{4}n^2+11)$	$\times W$	$\frac{1}{12}$ L sec α
Bb	$+\frac{1}{\sqrt{n^2+4}}(\frac{31}{12}n^2+9)$	$\times W$	$\frac{1}{12}$ L sec α
Cc	$+\frac{1}{\sqrt{n^2+4}}(\frac{11}{4}n^2+7)$	$\times W$	$\frac{1}{12}$ L sec α
Df	$+\frac{1}{\sqrt{n^2+4}}(\frac{11}{4}n^2+5)$	$\times W$	$\frac{1}{12}$ L sec α
Eg	$+\frac{1}{\sqrt{n^2+4}}(\frac{31}{12}n^2+3)$	$\times W$	$\frac{1}{12}$ L sec α
Fh	$+\frac{1}{\sqrt{n^2+4}}(\frac{11}{4}n^2+1)$	$\times W$	$\frac{1}{12}$ L sec α
La	$-\frac{11}{4}n$	$\times W$	$\frac{1}{6}$ L $\sec^2\alpha$
Ld	$-\frac{9}{4}n$	$\times W$	$\frac{1}{6}$ L $\sec^2\alpha$
Li	$-\frac{3}{2}n$	$\times W$	L$(1-\frac{1}{3}\sec^2\alpha)$
ab, bc, fg, gh	$+\frac{n\sqrt{n^4+40n^2+144}}{6(n^2+4)}$	$\times W$	$\frac{1}{4}$ L $\sqrt{\frac{\sec^2\alpha}{9}+\sec^2\alpha\tan^2\alpha}$
de	$+\frac{3n}{\sqrt{n^2+4}}$	$\times W$	$\frac{1}{4}$ L sec α tan α
cd, ef	$-\frac{1}{2}n$	$\times W$	$\frac{1}{6}$ L $\sec^2\alpha$
ei	$-\frac{3}{4}n$	$\times W$	$\frac{1}{6}$ L $\sec^2\alpha$
hi	$-\frac{5}{4}n$	$\times W$	$\frac{1}{6}$ L $\sec^2\alpha$

Coefficients for Calculating Lengths of Truss Members

Values of n	3	24/7	2 cot 30°	4	24/5	5	6
Values of α	33°41′24″	30°15′23″	30°	26°33′54″	22°37′12″	21°48′5″	18°26′6″
sec α	1 2018	1.1577	1 1547	1 1180	1.0833	1 0770	1.0541
$\sec^2\alpha$	1 4444	1.3403	1 3333	1 2500	1.1736	1.1600	1 1111
sec α tan α	0 8012	0.6753	0 6667	0 5590	0.4514	0 4308	0 3514
$\sqrt{\frac{\sec^2\alpha}{9}+\sec^2\alpha\tan^2\alpha}$	0 8958	0 7778	0 7698	0 6718	0 5781	0 5608	0.4969

Member	n = Span ÷ Height = 2 cot α						
	3	24/7	2 cot 30°	4	24/5	5	6
Aa	6 31	6.95	7 00	7.83	9.10	9.42	11.07
Bb	5 76	6.44	6 50	7 38	8 72	9 05	10 75
Ce	5 20	5.94	6 00	6 93	8.33	8 68	10.43
Df	4.65	5.43	5.50	6 48	7 95	8 31	10 12
La	5.25	6.00	6.07	7 00	8.40	8 75	10.50
Lc	4.50	5.14	5 20	6 00	7 20	7 50	9.00
Lg	3 00	3.43	3 46	4.00	4.80	5.00	6 00
ab, ef	0 83	0 86	0 87	0 89	0.92	0 93	0 95
cd	1 66	1.73	1 73	1 79	1 85	1.86	1 90
bc, de	0.75	0 86	0 87	1.00	1 20	1 25	1 50
dg	1 50	1 71	1 73	2.00	2 40	2 50	3.00
fg	2.25	2 57	2 60	3 00	3 60	3.75	4 50

Member	n = Span ÷ Height = 2 cot α						
	3	24/7	2 cot 30°	4	24/5	5	6
Aa	9.92	10 91	11.00	12 30	14 30	14 81	17.39
Bb	8.95	9 91	10.00	11 25	13.18	13 66	16 13
Cc	8.81	9.91	10.00	11.40	13.53	14 07	16.76
Df	8 25	9.40	9.50	10 96	13 15	13 70	16.44
Eg	7 28	8.41	8.50	9.91	12 02	12 55	15.18
Fh	7.14	8 40	8.50	10.06	12 38	12.95	15 93
La	8.25	9 43	9.53	11 00	13 20	13.75	16.50
Ld	6.75	7.71	7.79	9.00	10.80	11.25	13.50
Li	4.50	5.14	5.20	6.00	7.20	7.50	9.00
ab, bc, fg, gh	0 93	0 99	1.00	1 08	1 18	1.21	1.34
de	2 50	2.59	2.60	2.68	2 77	2.79	2.85
cd, ef	1.50	1 71	1 73	2.00	2 40	2 50	3.00
ei	2 25	2.57	2 60	3 00	3.60	3 75	4.50
hi	3.75	4.29	4 33	5 00	6 00	6 25	7.50

The pitch of a truss is the ratio of the rise or height to the span length of the truss. Pitch = H/L = 1/n. n = L/H = 1/pitch.
To obtain the stress in any member of a given truss, multiply the corresponding coefficient by the panel load W.
Compression members are designated by + and tension members by −.

Fig. 305*

* Based upon the "Pocket Companion," Carnegie Steel Co.

when a truss has varied loads with possibly some applied to the bottom chord, the usual coefficient table will not correspond with the problem under consideration. One does not learn how to analyze truss action by their use and the calculation of stresses becomes a mechanical process. The tables do offer an excellent means, however, for checking certain truss solutions.

Some engineers prefer to use a method in determining stresses which corresponds with that of coefficients to some extent. They use 1# panel loads and determine the coefficients for all the members on this basis, by any of the methods previously discussed. The coefficients can later be multiplied by the panel loads, such as for dead load, and again for snow load. An advantage of this method is that smaller figures are involved in the computations, and it is also easier to obtain the end reactions and the value of the shear in any panel. In general, this method is more commonly used in truss solutions when the truss is subjected to moving loads.

Prob. 193a. Check the coefficients shown in the left of the diagram in Fig. 305.

194. Maximum Stresses.

The maximum stresses which may occur in a truss for any length of time must be determined before the members can be designed. The loads caused by dead weight, snow, and wind, as well as the methods of obtaining the stresses resulting from such kinds of loading, have been previously discussed. It remains to determine what combination of these stresses should be used to obtain a reasonable maximum in each case.

It is not a reasonable assumption to add the total stresses due to each kind of loading. This would infer that a maximum depth of snow would remain undisturbed on a roof even when the wind reached a very high velocity. In view of this feature, careful judgment must be exercised to determine the probable maximum condition. The dead load is always present so that the stresses due to such load are always acting. It then becomes necessary to anticipate the combination of wind and snow which will exist for any length of time as a maximum.

When no wind is blowing, a full snow load may exist. **Hence one case is that of dead load and full snow load.** When the wind acts upon the truss, it will tend to blow some of the snow off, if any is present. Another condition may be that of very wet or frozen snow which remains undisturbed by the wind. In the latter case, such snow would very seldom exist at a maximum depth, and a fair assumption is that half of the maximum snow load might exist. It should be remembered that the wind load is assumed to act perpendicularly to the roof surface and that it exerts no pressure on the leeward side. **A second combination** on the basis of the above reasoning is to obtain the stresses due to **dead load, one-half snow load, and full wind load.** The wind can blow from only one direction at once so that the wind load stress to be added is the larger of the two resulting from the wind on the left and that on the right. Some designers assume a condition of dead load, maximum snow load on the leeward side of the truss and full wind load, as another possibility. This assumption does not seem reasonable because the wind when passing over the peak of a roof probably causes a reduction of pressure and sets up eddy currents on the leeward side which tend to remove some of the snow.

Another possibility which may occur, however, is that of full snow load and a wind force which is insufficient to remove any appreciable amount of snow. Such an amount would probably not exceed one-third of the maximum wind load. **A third combination is then dead load, full snow load and one-third wind load.**

It does not seem wise, as stated above, to provide for extreme conditions. For instance, a very heavy snowfall might occur, followed by a slight rainfall, then a sudden freezing, followed by a high wind. The probability of these all happening in just that sequence is remote. The duration of the load is also an important factor. It is an established fact that members can withstand a higher stress for a short time than they can for a considerable period. Another consideration is that the snow on steep roofs may slide off before any appreciable amount of wind load acts. To summarize the preceding discussion, the three combinations for maximum stress commonly investigated are as follows:

(1) dead load + full snow load,
(2) dead load + $\frac{1}{2}$ snow load + full wind load, and
(3) dead load + full snow load + $\frac{1}{3}$ wind load.

Some designers prefer to use only the first two conditions. In localities where the snowfall is an average, (2) and (3) are nearly equal. Where the snowfall may be heavy, either (1) or (3) will give a maximum. For certain types of trusses (as discussed in Art. 151) a combined vertical loading is sufficiently accurate for stress analysis. For all other types of trusses, each condition should be tested out to determine the maximum stress in each member. Figure 306 illustrates a table heading which is convenient for this work. When (3) is considered, (1) need not be. One other consideration which is made for some trusses is to consider the dead load, full snow load on one side only with the wind blowing on that side.

This is of course a possible condition. In unusual instances, such loading may cause a reversal of stress. If a member which is subjected to tension by ordinary loading has compression developed in it by this special case of loading, care must be exercised to design the member for the worse condition.

T = the maximum tension in lbs. in the member, and

f_t = the maximum allowable tensile stress in lbs. per sq. in.

The length of the compression members is usually such that a formula similar to the above cannot be

TABLE OF MAXIMUM STRESSES IN 1000*											
Truss Member	Dead Load.	Snow Load		Wind Load		Wind Load		Combinations			Maximum
		Full	$\frac{1}{2}$	Leew.	Windw.	Max.	$\frac{1}{3}$ Max.	D.L+S.L.	D.L+½S.L. +Max.W.L.	D.L+S.L. +⅓W.L.	
B-1 etc.	38.5	35.1	17.6	22.5	22.4	22.5	7.5	73.4	78.6	83.1	83.1

FIG. 306

195. Maximum Reactions.

In order to make a complete design of a truss, it is necessary to obtain the maximum values of the end reactions. These may be determined by assuming the same conditions as those for the maximum stresses. The latter are commonly limited to

(1) dead load and maximum snow load, or
(2) dead load, one-half snow load, and maximum wind load.

The half panel loads at the ends of the truss must be considered to obtain the maximum end reactions. Such vertical loads do not affect the stresses in the truss members, but are obviously a part of the load which must be carried by the supports.

It is usually most convenient to determine the reactions in terms of their horizontal and vertical components. The vertical force must be sustained by bearing upon the support and the horizontal force by some other means of horizontal resistance. (See Index.)

196. General Design of Truss Members.

The design of the usual truss member is based upon the assumption that the stress developed in it is axial, that is, acting along the length of the member. This involves only direct stresses, namely, tension or compression. The former requires a sufficient net section to transmit the stress at a safe working value, and the formula

$$A_N = \frac{T}{f_t} \qquad (S\text{-}52)$$

may be used in which

A_N = the required net area in sq. ins. (exclusive of rivet or bolt holes),

used on account of the common limiting ratios of slenderness. In other words, the maximum allowable stress, p, must be obtained from a formula which provides against the sidewise bending induced by column action. The effective length is theoretically considered as the distance between joints in the ordinary truss. This is based upon the assumption that the members are held rigidly to the plane of the truss by the joints. The top chord, which is the most important compression member, is stiffened in a sidewise direction by the purlins. In practice, the top chord is continuous over one or more panel points.

When loads are applied between the panel points of the chord members, the design must not only provide for the direct stress in a member, but in addition, should provide for the indirect stress due to cross bending induced by the intermediate loads. The combined stress must not exceed the specified limit. Some specifications allow a maximum which is 25% in excess of the usual working stress. Long tension members (especially when horizontal) should be supported at intervals in order to relieve them of the extra stress caused by their own weight. For this reason "sag ties" are often introduced at the center lines of trusses to aid in supporting the bottom chord, and to prevent excessive deflection or stress. Since comparatively little stress is developed in a sag tie, it is made a minimum size.

Another assumption in the truss theory is that the centers of gravity of the cross sections of the members are coincident with the working lines of the truss diagram. This is not always practically the case, and for any considerable variations, the member should be designed to sustain in addition the indirect stress induced by the moment of the direct stress times the eccentricity of its line of action.

197. General Design of Joints.

The basic principle involved in the design of the joints is that the stress in any member which terminates at a joint must be developed by the fastening. Thus in Fig. 307 (*a*) the stresses in 4–5 and 5–6 must be resisted by their fastenings. If the horizontal member is continuous by the joint, only

FIG. 307

the difference of the stresses needs to be considered. If a break in the horizontal member occurs, as in (*b*), then the stress either side of the joint must be provided for. The nature of the details varies greatly with the kind of truss material used. The design of truss joints is more fully discussed later.

198. Truss Bracing.

A well designed truss is sufficiently stiff in the direction of its length, but it should be braced to prevent its tipping sidewise. This bracing helps to overcome the tendency of racking, as illustrated in Fig. 308 (*a*). The type of bracing, and its extent, depend upon the character and the use of the building. The effects of the wind, vibration due to moving loads or machinery in operation, and the facilities of field erection are some of the important factors which must be considered. In other words, the purpose of bracing is to transmit all lateral forces as directly as possible to the walls and thence to the ground. Since such lateral forces may be exerted in any direction, bracing is commonly provided by two definite systems, namely, longitudinal and transverse.

Longitudinal bracing may be classed into three types:

(1) lateral, which extends at right angles to the plane of the truss,

(2) sway, which is made up of diagonals placed between the laterals, and

(3) side, which is used between the exterior columns in the side walls of a framed building.

These are illustrated in Fig. 308. The **lateral and sway bracing** are often grouped as one system. It may be placed either in the plane of the top chord or in the plane of bottom chord, or both. Since compression members are more susceptible to sidewise bending, the plane of the top chord is the more important one to brace. Top chord bracing must be sufficient to resist the lateral forces developed by the wind. Bottom chord bracing, if any is used, is generally lighter, as its prime purpose is to absorb vibration and to aid in erection.

Transverse bracing is used only when the trusses are supported by columns. It is absolutely necessary in many cases because the columns depend upon such bracing. It is almost always provided in the form of knee braces extending from the bottom chord of the truss to the face of the supporting columns, as illustrated in Fig. 308 (*d*). The angle α is sometimes made 45°.

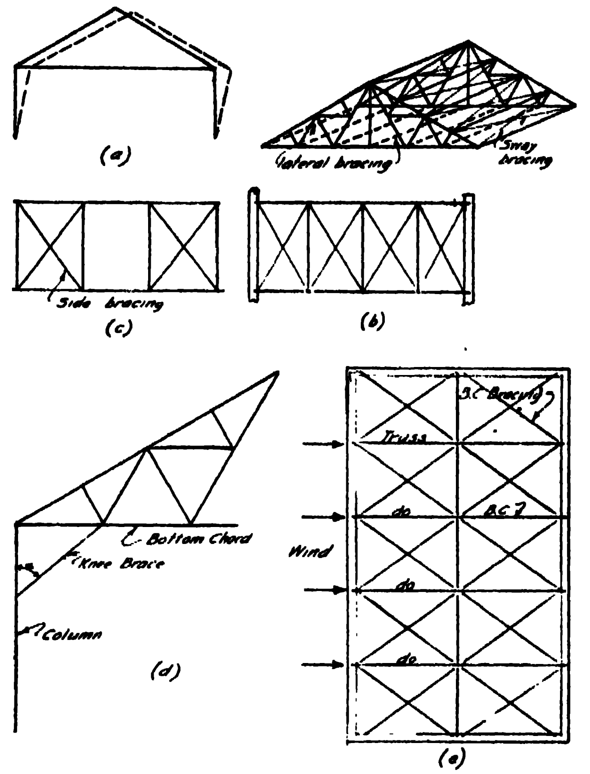


FIG. 308

It is unnecessary to provide full systems of bracing for all kinds of roofs. When trusses rest upon masonry walls, the purlins and roof carrying materials are usually sufficient to brace the roof adequately. If there is a ceiling frame, this helps to brace the bottom chord. When machinery, cranes, and the like, are to be used in a building, bracing is generally provided in the plane of the bottom chord to absorb any vibration. In hipped roofs, the hip trusses, purlins, sheathing, and ceiling beams provide sufficient bracing. If the roof terminates in gable walls, it is often braced in the end bays by diagonals from the purlins to the ceiling frame, placed close to the gables. In such cases the purlins and ceiling framing must be well anchored. In an extremely long building there is some danger of the longitudinal walls tending to

bulge at the middle of their lengths, due to wind action. This is offset by providing a "horizontal truss" to transmit the pressure from the dangerous point to the end walls. This "truss" in reality has the longitudinal walls for chords and the bottom chords of the roof trusses as "verticals" in the horizontal plane. The "web system" of such a "truss" is supplied by diagonals between the roof trusses in the plane of their bottom chords. These are in reality bottom chord bracing. The "truss" action is illustrated in Fig. 308 (*e*).

When it is necessary to provide top chord bracing, it is always placed in the last bays at each end of the building at least. This ties the first interior truss to the wall and aids in the erection. This is theoretically all that is necessary to resist the wind forces exerted upon the ends of the building, as these will be dissipated by the panel of bracing transferring them to another truss. However, in long buildings, the trusses should be braced in pairs so that each panel of top chord bracing is not more than three or four bays apart. Such pairs of trusses should be tied together. This is usually accomplished by the use of a "ridge strut" extending between all trusses and in the plane of the top chord at the mid-span.

The actual stresses which determine the design in bracing may be classed as indeterminate, due to the many conditions of loading. For this reason the provision of bracing is largely a matter of judgment developed by a study of mechanics and by experience gained in design and construction. However, certain stresses may be approximated by making assumptions (see Index — Truss Bracing).

199. Future Extension.

If there is a possibility that the building being designed may be extended at some future time, a typical truss should be provided in the end wall. In such a case the latter is a curtain wall instead of a bearing wall. In this manner much labor and cost will be saved when the extension becomes a reality.

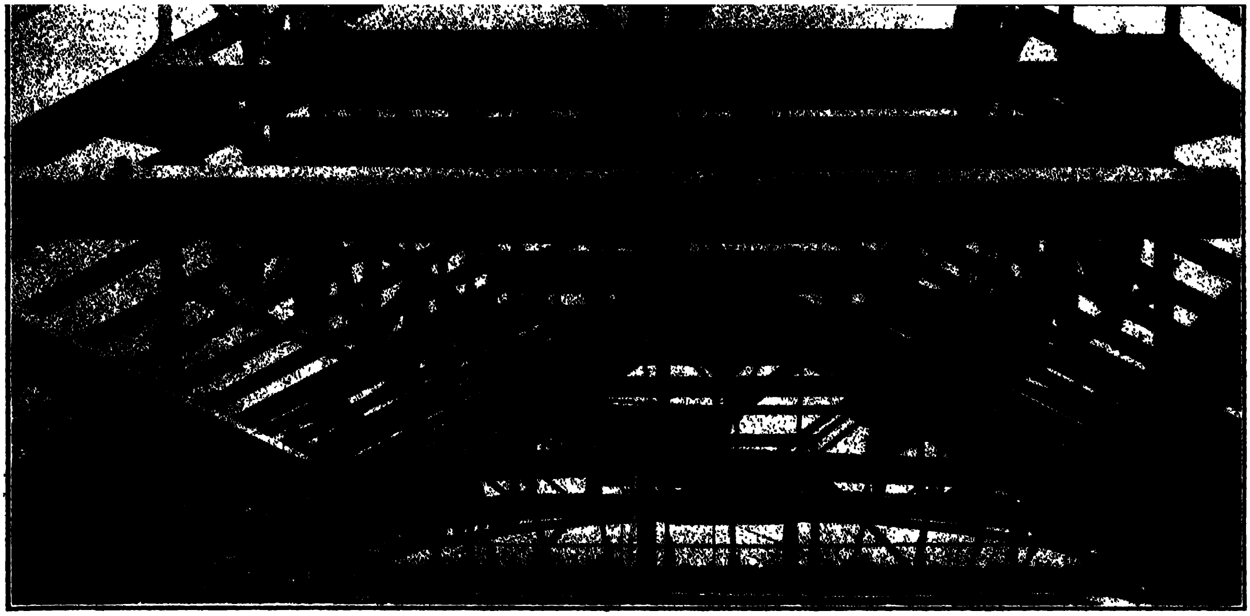


PLATE 26 TYPICAL TRUSSES IN PLACE

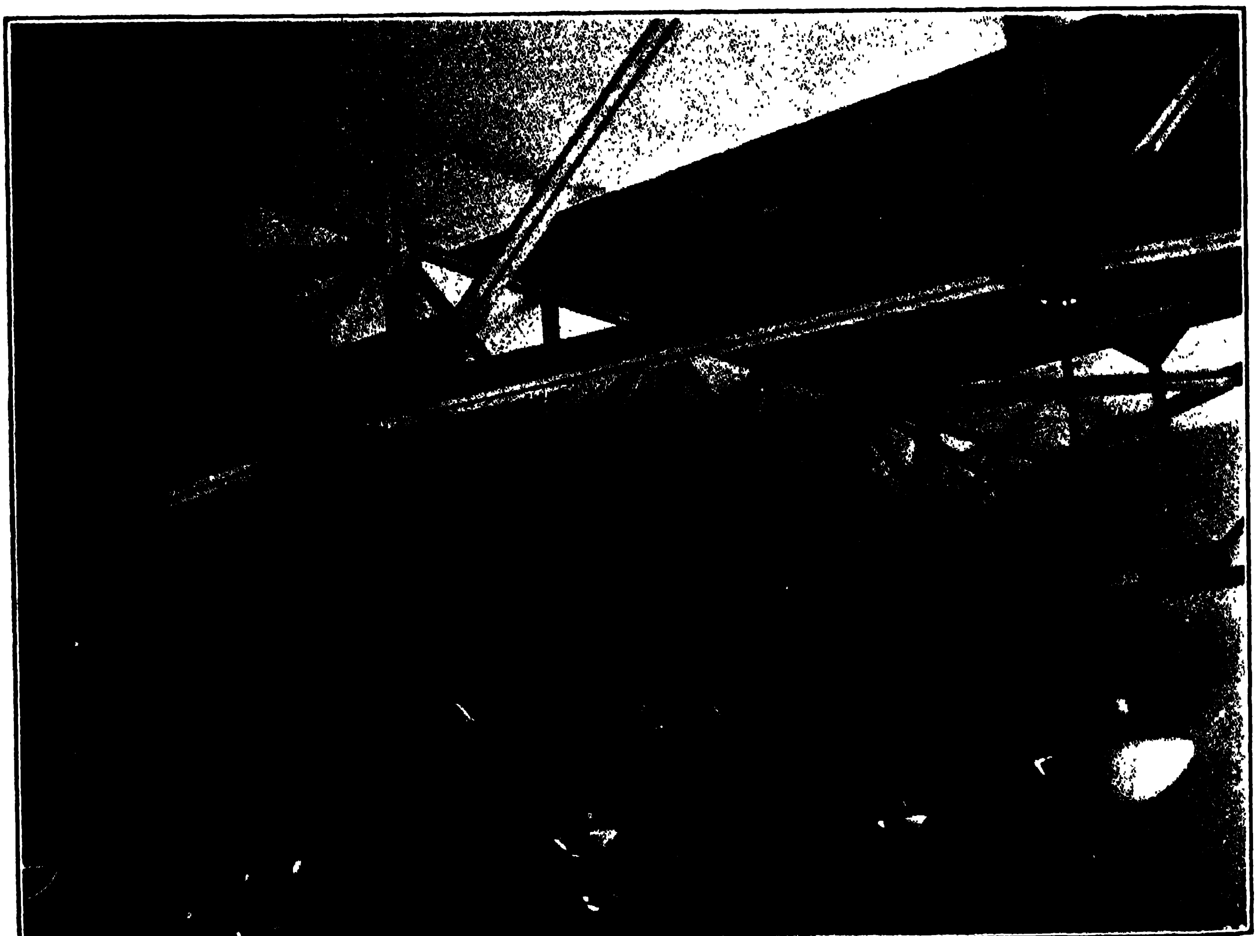


PLATE 27 TOP—TRUSS READY FOR SHIPMENT
BOTTOM—EXPOSED TRUSSES IN A FINISHED CEILING

CHAPTER 16

COMMON TYPES OF STEEL TRUSSES

200. General Considerations.

In modern practice, roof trusses are most commonly made of structural steel shapes because the fabrication is more positive, cheaper, and less complicated than that of wood trusses. The fundamental principles of their design have already been discussed (Chap. 15), and only the additional features common to steel trusses need to be considered further. The structural sections used, usually angles, are connected at their natural points of theoretical intersection by what are called **gusset plates.** These get their name from the similarity to the gussets used in tailoring at the armpits; in fact the word derives its origin from the French in exactly this manner.

201. Types of Ordinary Steel Trusses.

One of the most common types of steel frames is that shown in Fig. 309 (*a*), which is called a Fink truss. This is of course adaptable to the usual V,

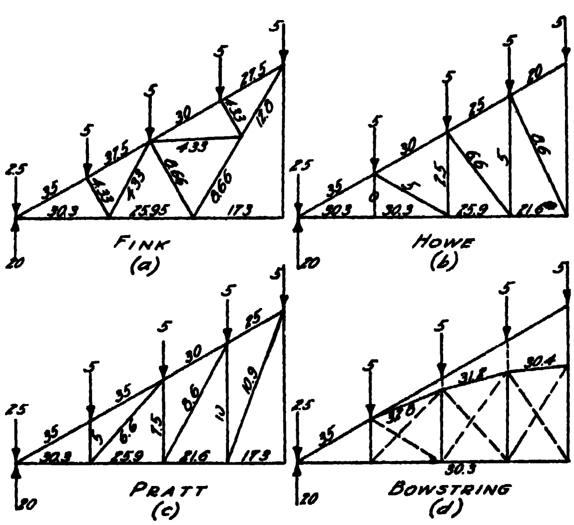


FIG. 309

or pitched, roof. Other types of trusses are also shown in the figure, — the stresses being represented for the same conditions of span and loading in each. The type in (*b*) is a Howe truss, and that in (*c*) is a Pratt truss. The maximum stresses in the top and bottom chords are the same in each, although the values vary, as noted in the figure, toward the center of the span. The top chord stresses decrease more rapidly in the Howe truss than in the Pratt type, while the reverse is true for the bottom chord stresses. The web stresses in general are less in the Howe truss. However, the longer web members, or the diagonals, are in compression in this type. Timber sections are not particularly adaptable to tensile stresses, while steel is an ideal material in tension. On the other hand, wood is comparatively well adapted to compression, and steel is not as economical relatively. For these reasons, **the Howe truss is adapted to wood framing and the Pratt type is preferable for steel trusses.**

Figure 309 (*d*) shows a Bowstring truss. The variation in the top chord stresses is small and the stress in the bottom chord is constant. This simplifies their design. The top chord approaches the slopes of the strings drawn in the equilibrium polygon, as illustrated, when the coincidence is exact, the stresses in the diagonals are theoretically zero. In practice, however, diagonals would be used to protect against unbalanced loads. This type of truss is not very common for a V-roof, as extra scantlings are required for furring to form the roof pitch, as shown. The rigidity of these is usually none too good.

The Fink truss offers advantages over the others which make it a desirable type. The web struts are perpendicular to the top chord and this allows simple framing details. These struts are also in short lengths, and, therefore, require less sectional area. The appearance of the truss as a whole is better than that of the Pratt type. The similarity of the web members, and the equality of the stresses in corresponding members allows duplication of sizes. A large number of the members are in tension, which is a distinct advantage. For these reasons, **the Fink truss is easily adapted to structural steel framing,** and it is more economical than a similar Pratt truss, except when light roof loads and no ceiling loads occur.

Fink trusses are generally made with an even number of panels. Figure 310 shows possible variations in the number used, this depending upon the span and other practical conditions of a given problem (Art. 158). When vertical struts are used instead of those perpendicular to the top chord, the

frames are called **fan trusses.** In some cases, it is desirable to divide the top chord into shorter lengths to provide supports for the purlins at closer spacings. This is sometimes called a truss with "sub-divided" panels, or the truss is said to be "compound" as in (*e*) and (*f*). When trusses on a long span have a horizontal bottom chord, an optical illusion occurs, in that the truss appears to sag. When the truss is exposed to view, it is sometimes **cambered,** as shown in (*g*) and (*h*), in which case

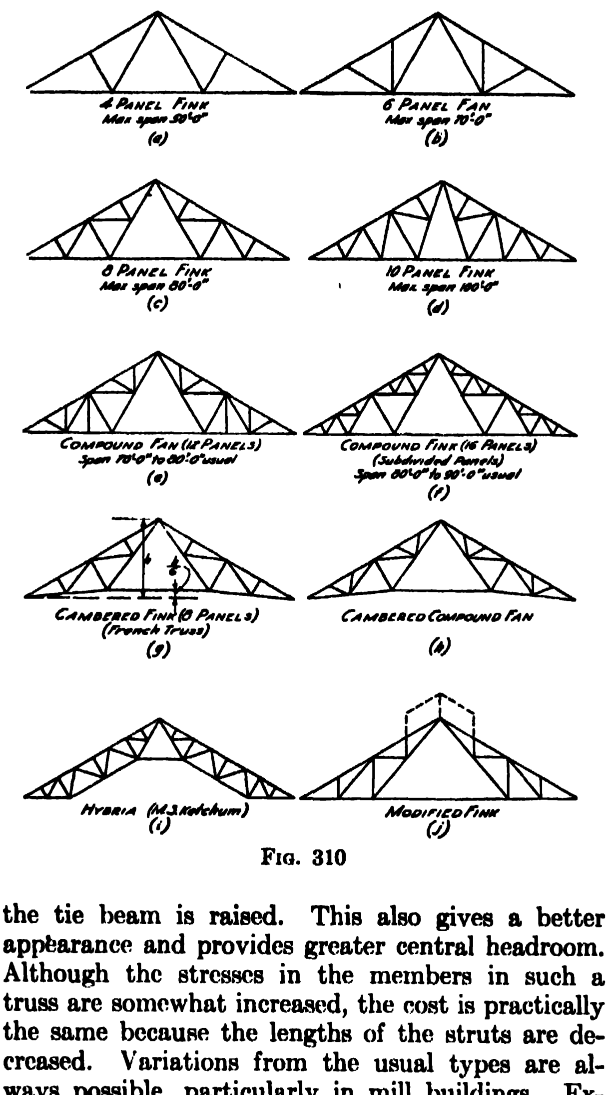


FIG. 310

the tie beam is raised. This also gives a better appearance and provides greater central headroom. Although the stresses in the members in such a truss are somewhat increased, the cost is practically the same because the lengths of the struts are decreased. Variations from the usual types are always possible, particularly in mill buildings. Examples are shown in Fig. 310 (*i*) and (*j*). In such structures, monitors are common, as indicated by the dotted lines in (*j*).

Pratt truss types may also be varied as to the number of panels used, as illustrated in Fig. 311 (*a*). For steep pitched roofs, the types illustrated in (*b*) are quite common. In summary, the type of the truss must conform to the requirements of the roof void, as well as to the span and arrangement of the secondary roof framing. General types of trusses for deck, mansard, gambrel, and flat roofs are discussed and illustrated in Art. 157

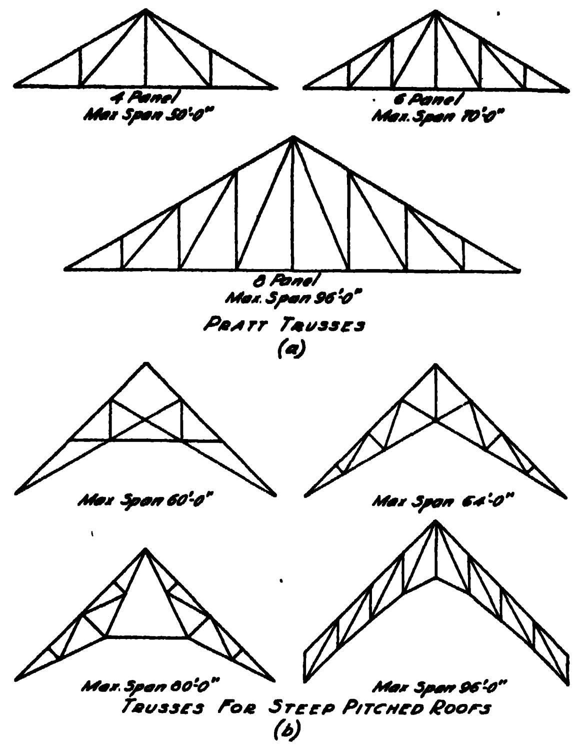


FIG. 311

202. Weights of Trusses.

In determining the loads that a truss is to carry, an allowance should be made for the weight of the truss itself. This is most commonly done by allowing an assumed number of #/□′ of horizontal projection, and adding this value into the other unit dead loads. The factors which influence the weight of a truss are the span, spacing of trusses, and to some extent the pitch, although pitches from $\frac{1}{4}$ to $\frac{1}{2}$ have little effect when the bottom chord is horizontal. If the pitch is much less than $\frac{1}{6}$, the weight increases more rapidly. The weight of trusses with cambered bottom chords increases from 15% to 40% over that with horizontal bottom chords. The usual formulas contain the above variables in some form. There are a number of formulas which may be used to approximate a value to use in making computations. Some of the more common of these are:

$$w = 0.75 + 0.075\,L \quad \text{(Merriman, and Howe)}, \qquad (S\text{–}44)$$

$$w = \frac{L}{25} + \frac{L^2}{12{,}600} \quad \text{(Ricker)},$$

$$w = 0.5\,(1 + 0.15\,L) \quad \text{(Jacoby), and}$$

$$w = \tfrac{1}{8}\,(\sqrt{L} + \tfrac{1}{8}\,L) \quad \text{(Carnegie), in which}$$

w = the weight of a truss in #/□′ of horizontal projection, and

L = the span of the truss in feet.

The following table (Table 71) may be used as an alternative in determining the allowance.

TABLE 71
WEIGHTS OF STEEL TRUSSES
(#/□′ of Horizontal Projection)

Span (Ft.)	Pitch			
	½	⅓	¼	Flat
Up to 40	5.2	6.3	6.8	7.6
50	5.7	6.6	7.2	8.0
60	6.7	8.0	8.6	9.6
70	7.2	8.5	9.2	10.2
80	7.7	9.0	9.7	10.8
100	8.5	10.0	10.8	12.0
120	9.5	11.0	12.0	13.2
140	10.0	11.6	12.8	14.0

203. Determination of the Stresses.

In the usual instances of steel truss design, the graphical method is commonly used (Art. 164). Figure 312 shows a typical solution of this kind for vertical loading on a Fink truss. This is one of the most common types of steel trusses. One special feature must be considered in its design. When the joint C-D is reached in the solution, the stresses in the members beyond seem indeterminate at first, because there are three unknown forces at the joint. This situation may be avoided by drawing in the imaginary member from x in the space 4-5-6, and considering it as a replacement of the members 4-5 and 5-6. This is permissible, as the truss is still an assemblage of rigid triangles. The stresses in the remaining members are the same regardless of the web system to the left. These may then be determined in the usual way, assuming that the members 4-5 and 5-6 are not in the truss for the time being. Thus the lines 3-k and d-k determine the point k in the force diagram, and then the lines k-j and e-j fix the point j (point 6). Then 6-5 and d-5 determine point 5, and 5-4 and 3-4 fix point 4. The line 6-7 and the line a-7 produce the intersection at 7. The point 4 should, of course, be on the line 6-7 in order to check the solution. The stresses are now determined on the basis of the real members. An alternate solution may be made by calculating the value of the stress in a-7 (Art. 165) and plotting it on the diagram. The latter method is simpler than the first. Care must be taken that the value is correctly computed, however, or the diagram would not close and would, therefore, be in error.

Wind load stresses may be determined by the methods discussed in Arts. 163 and 164, and the maximum stresses established (Art. 169).

Prob. 203a. Determine the maximum stresses in the typical truss shown in Fig. 298–12 for the following data:

Span 60′-0″. Pitch ¼. Slate roof, 2″ plank, steel purlins spaced in equal panels. Snow load 20#/□′. Wind load (horizontal direction) 30#/□′. Spacing of trusses 14′-0″.

Prob. 203b. Determine the values of the stresses in all the members of the truss shown in Fig. 313 (a). Use graphical method.

Prob. 203c. Check the results in Prob. 203 (b) by an analytical solution.

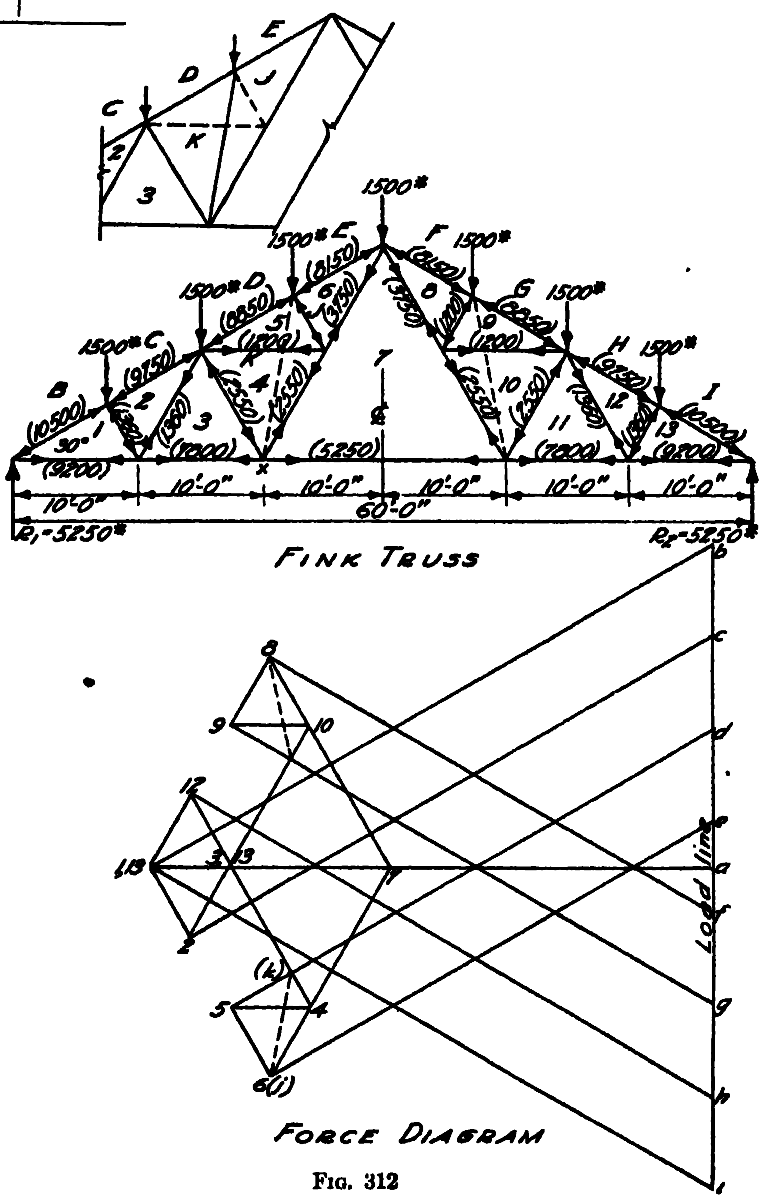


Fig. 312

Prob. 203d. Determine the stresses in the truss shown in Fig. 313 (*b*).

Prob. 203e. Determine the stresses in the truss shown in Fig. 313 (*c*).

Prob. 203f. Establish the values of the maximum stresses for the truss of Fig. 313. Use the following combinations:

(*a*) D.L. + ½ S.L. + max. wind, and
(*b*) D.L. + full S.L.

Snow panel load = 1800#.

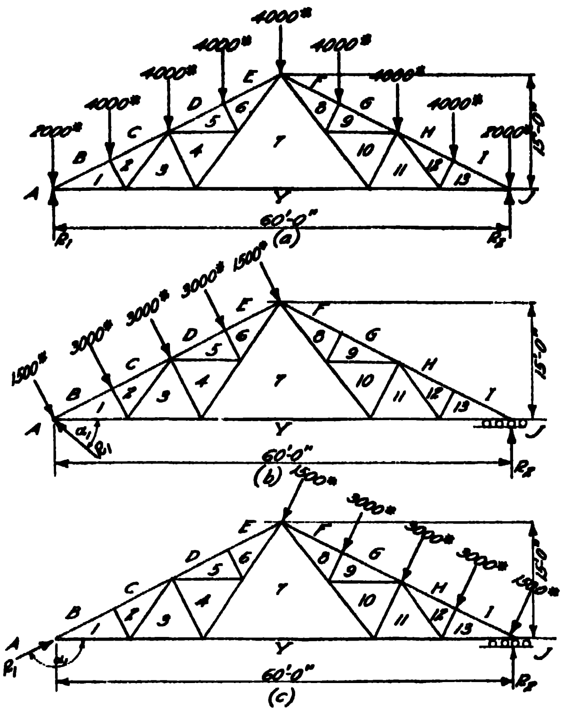


FIG 313

204. Common Sections Used.

When the maximum stresses in the members of a truss have been established, the next step is to determine the required sizes. This involves two types of problems, namely, that affecting members carrying tension and that affecting members resisting compression. In both instances, however, rolled structural **steel angles are used in the large majority of cases*** although a pair of channels is occasionally used for a bottom chord. Angles are generally cheaper, easier rolled, and easier handled than other structural shapes. **The principal members should always be made of two angles,** placed back to back, and separated by a space corresponding to the thickness of the gusset plates. A single angle is undesirable for a strut because it tends to buckle in its weakest direction, which is about a diagonal axis. Such a selection requires more sectional area than a pair of smaller angles. A twisting moment is also introduced at the ends of the member, which tends to tear the section through the rivet holes. If, in a special case, a single angle is used for a compression member, it should be an equal-legged angle, as the greatest radius of gyration is available for a given area. In small trusses, single angles may be used for web members acting in tension. In such cases, these single members should be staggered on opposite sides of their respective gusset plates, to give "balance" to the truss. For large trusses, however, all members should be composed of two angles. **The angles are generally unequal legged, with the short legs outstanding,** because greater stiffness for the given area may be obtained.

Minimum dimensions are established for practical reasons. For light trusses, the web angles are sometimes made $\frac{1}{4}''$ minimum thickness. For chord members and the web angles of larger trusses, the least thickness is commonly $\frac{5}{16}''$. This is also the thinnest angle available in some of the larger sizes, and $\frac{3}{8}''$ is the minimum for large angles (Table 9). When trusses are to be exposed to the weather or to corroding gases, $\frac{3}{8}''$ is often specified as a minimum thickness. The smallest-sized leg in which a $\frac{3}{4}''$ rivet can be driven is $2\frac{1}{2}''$ (Table 21). Small web members are often attached to the gusset plates by rivets through one leg only. Since $\frac{3}{4}''$ rivets are the common fastenings for ordinary trusses, **the minimum size of angle is usually** $2\frac{1}{2} \times 2 \times \frac{1}{4}$. When clip angles are used and riveting occurs in both legs of the member, $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$ is the minimum. **The angles** used in all trusswork **should conform to stock sizes** (Art. 5).

In contrast to the pair of angles for the chord members shown in Fig. 314 (*a*), a pair of channels, as in (*b*), may be used. This is usually done when the stresses are large, or when the chord carries a heavy uniform load. The channels also furnish convenient supports for ceiling joists or rafters. For small stresses, angles may be used to reduce the truss weight. Two angles and a plate, as illustrated in (*c*), may be employed for chord members if desired, especially when large stresses are encountered. Such a combination saves some gusset plate work, but field painting is not done as efficiently as when angles or channels are used alone. More rivets are also necessary when continuous plates are used, and they should not be spaced more than 16 times the thickness of the thinnest metal nor more than 6" apart.

205. Design of Tension Members.

The design of steel tension members involves the simple provision of ample net sections to resist

* Flat bars are occasionally used for tension members in certain types of trusses. The only advantages which may be claimed are a smaller amount of riveting and a smaller weight. They are not desirable for members subjected to a reversal of stress nor for compression members. A pair of channels or a pair of angles and a plate are occasionally used for top chord or bottom chord sections, as discussed later.

the stresses, which should not exceed the maximum allowable unit stress provided in the code. This means that an allowance for rivet holes must be made (Art. 21). **Under ordinary circumstances, one hole per angle per section is sufficient.** In order to include the weakening effect of only one

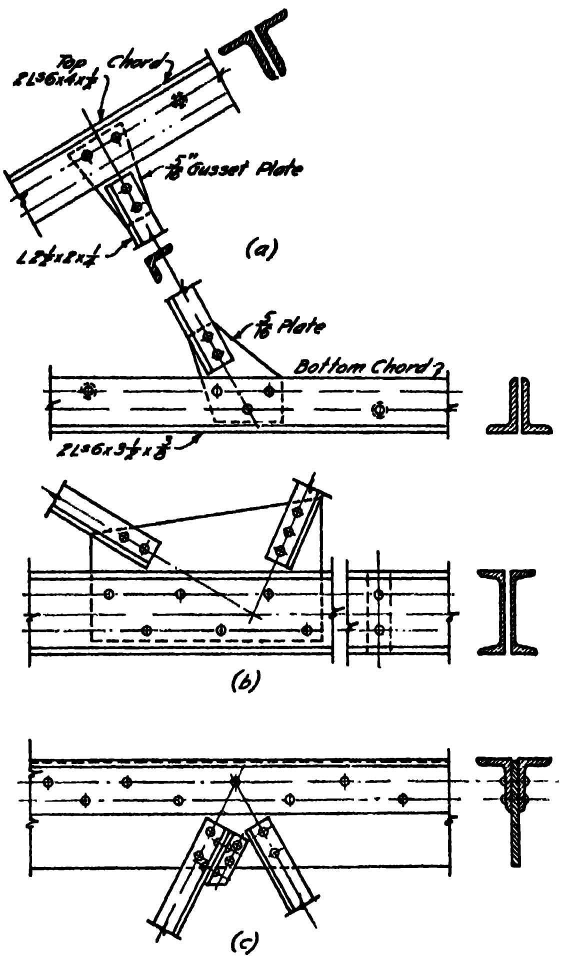


Fig. 314

rivet hole, care must be taken to provide ample stagger of the rivets in the different gauge lines to **maintain the net section** (Art. 48). The two angles selected should have a gross area sufficiently in excess of the theoretical required area so that when the holes are deducted, a small excess still remains. The smallest sized angle which it is possible to use is not generally the most economical, particularly when large stresses are involved. Usually an angle with wider legs and of thinner material will weigh less per foot. Some study of comparative sizes is generally valuable in this respect, particularly when there is considerable duplication in the trusses. The maximum allowable tensile stress for structural steel shapes is commonly specified as 16,000 #/□", although some codes have recently increased this to 18,000 #/□".*

Illustrative Prob. 205a. Select a pair of angles to carry a tension of 64,000#. Use $\frac{3}{4}$" rivets.

$\frac{64{,}000}{16{,}000}$ = 4.00□", net area for 2 L^s, or

2.00□", net area for 1 L.

Try $3\frac{1}{2} \times 2\frac{1}{2}$ L. Area $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{7}{16}$ = 2.43□"
1 Hole out = $\frac{7}{8} \times \frac{7}{16}$ = 0.38
Net area = 2.05□"
Weight = 8.3#/ft.

Try 4×3 L. Area $4 \times 3 \times \frac{3}{8}$ = 2.48□"
1 Hole out = $\frac{7}{8} \times \frac{3}{8}$ = 0.33
Net area = 2.15□"
Weight = 8.5#/ft.

Try 5×3 L. Area $5 \times 3 \times \frac{5}{16}$ = 2.40□"
1 Hole out = $\frac{7}{8} \times \frac{5}{16}$ = 0.27
Net area = 2.13□"
Weight = 8.2#/ft.

Use 2 L^s $5 \times 3 \times \frac{5}{16}$.

Illustrative Prob. 205b. Select a pair of angles to carry a tension of 8000#.

$\frac{8000}{16{,}000}$ = 0.5□" net area required.

A minimum angle is amply sufficient.

Use 2 L^s $2\frac{1}{2} \times 2 \times \frac{1}{4}$.

The calculation of the tensile capacity of a single $2\frac{1}{2} \times 2 \times \frac{1}{4}$ L and also of two L^s $2\frac{1}{2} \times 2 \times \frac{1}{4}$ will give a convenient maximum to bear in mind. When tensile stresses in truss members are less than such values, the sizes may thus be arbitrarily established. Thus the value of 1 L $2\frac{1}{2} \times 2 \times \frac{1}{4}$ with one $\frac{3}{4}$" rivet hole out is $(1.06 - 0.875 \times 0.25)$ 16,000 = 13,400#. That for a pair of such angles is obviously 26,800#. Since the safe tensile capacity, with a definite size of hole out, for a given angle is always the same, a table of such strengths is a convenience and time-saver in making computations. Table 72 gives values of this kind for common sizes of angles.

Prob. 205c. By calculations, select a pair of angles to carry a tension of 106,000#. Use $\frac{7}{8}$" rivets, one hole out of each angle. Use 18,000#/□" as a unit stress.

Prob. 205d. By calculations, select a pair of angles to carry a tension of 40,000#. Use $\frac{3}{4}$" rivets, one hole out of each angle. Use 16,000#/□" as a unit stress. Check your result by Table 72.

Prob. 205e. From Table 72, select a pair of angles to carry a tension of 30,000#. Is the pair selected of minimum weight? Select a single angle to carry the stress. Which combination weighs more?

206. Design of Compression Members.

The design of the compression members in a steel truss involves the use of a steel column formula (see Index). A generally accepted formula of this kind is:

* Again, the authors wish to reiterate that they believe 18,000#/□" is rather liberal, when full live loads occur, as the real factor of safety becomes less than 2.

TABLE 72*

ALLOWABLE TENSILE VALUES FOR ANGLES

(Thousands of Pounds)

Maximum Stress = 16,000 #/□″

Size	Thickness	Weight #/Ft.	Area □″	Net Areas and Stresses TWO HOLES DEDUCTED			
				$\frac{3}{4}''$ Rivs.		$\frac{7}{8}''$ Rivs.	
				Area □″	Stress	Area □″	Stress
8 × 8	1	51.0	15.00	13.00	208.0	13.25	212.0
8 × 8	$\frac{7}{8}$	45.0	13.23	11.48	183.7	11.70	187.2
8 × 8	$\frac{3}{4}$	38.9	11.44	9.94	159.0	10.13	162.1
8 × 8	$\frac{5}{8}$	32.7	9.61	8.36	133.8	8.52	136.3
8 × 8	$\frac{1}{2}$	26.4	7.75	6.75	108.0	6.87	109.9
8 × 6	1	44.2	13.00	11.00	176.0	11.25	180.0
8 × 6	$\frac{7}{8}$	39.1	11.48	9.73	155.7	9.95	159.2
8 × 6	$\frac{3}{4}$	33.8	9.94	8.44	135.0	8.63	138.1
8 × 6	$\frac{5}{8}$	28.5	8.36	7.11	113.8	7.27	116.3
8 × 6	$\frac{1}{2}$	23.0	6.75	5.75	92.0	5.87	93.9
6 × 6	$\frac{7}{8}$	33.1	9.73	7.98	127.7	8.20	131.2
6 × 6	$\frac{3}{4}$	28.7	8.44	6.94	111.0	7.13	114.1
6 × 6	$\frac{5}{8}$	24.2	7.11	5.86	93.8	6.02	96.3
6 × 6	$\frac{1}{2}$	19.6	5.75	4.75	76.0	4.87	77.9
6 × 6	$\frac{3}{8}$	14.9	4.36	3.61	57.8	3.70	59.2
6 × 4	$\frac{7}{8}$	27.2	7.98	6.23	99.7	6.45	103.2
6 × 4	$\frac{3}{4}$	23.6	6.94	5.44	87.0	5.63	90.1
6 × 4	$\frac{5}{8}$	20.0	5.86	4.61	73.8	4.77	76.3
6 × 4	$\frac{1}{2}$	16.2	4.75	3.75	60.0	3.87	61.9
6 × 4	$\frac{3}{8}$	12.3	3.61	2.86	45.8	2.95	47.2
5 × 3½	$\frac{5}{8}$	16.8	4.92	3.67	58.7	3.83	61.3
5 × 3½	$\frac{1}{2}$	13.6	4.00	3.00	48.0	3.12	49.9
5 × 3½	$\frac{3}{8}$	10.4	3.05	2.30	36.8	2.39	38.2
5 × 3½	$\frac{5}{16}$	8.7	2.56	1.93	30.9	2.01	32.2
5 × 3	$\frac{1}{2}$	12.8	3.75	2.75	44.0	2.87	45.9
5 × 3	$\frac{3}{8}$	9.8	2.86	2.11	33.8	2.20	35.2
5 × 3	$\frac{5}{16}$	8.2	2.40	1.77	28.3	1.85	29.6

$$p = 16{,}000 - 70\frac{l}{r}, \text{ in which}$$

p = the maximum allowable unit stress in #/□″, not to exceed 14,000,

l = the effective length of the member in inches, and

r = the least radius of gyration in inches.

Other column formulas may be used in accordance with code requirements and office standards. In the absence of other regulations the above expression is recommended. The ratio of slenderness, $\frac{l}{r}$, is usually limited in recognized practice.

SPECIFICATION CLAUSE

The effective or unsupported length of main compression members shall not exceed 120 times, and for secondary members 200 times, the least radius of gyration.

Truss members are considered to be main members. Those classed as secondary are members which serve as bracing, and the like.

As discussed in Art. 204, the most **common section** used for truss work **is a pair of unequal-legged**

* Based upon the "Pocket Companion," Carnegie Steel Co.

TABLE 72 — *Continued*

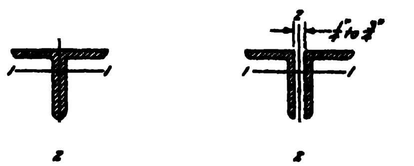
Size	Thickness	Weight #/Ft.	Area, □″	Net Areas and Stresses ONE HOLE DEDUCTED			
				$\frac{3}{4}''$ Rivs.		$\frac{7}{8}''$ Rivs.	
				Area □″	Stress	Area □″	Stress
6 × 6	$\frac{7}{8}$	33.1	9.73	8.85	141.6	8.96	143.4
6 × 6	$\frac{3}{4}$	28.7	8.44	7.69	123.0	7.78	124.5
6 × 6	$\frac{5}{8}$	24.2	7.11	6.48	103.7	6.56	105.0
6 × 6	$\frac{1}{2}$	19.6	5.75	5.25	84.0	5.31	85.0
6 × 6	$\frac{3}{8}$	14.9	4.36	3.98	63.7	4.03	64.5
6 × 4	$\frac{7}{8}$	27.2	7.98	7.10	113.6	7.21	115.4
6 × 4	$\frac{3}{4}$	23.6	6.94	6.19	99.0	6.28	100.5
6 × 4	$\frac{5}{8}$	20.0	5.86	5.23	83.7	5.31	85.0
6 × 4	$\frac{1}{2}$	16.2	4.75	4.25	68.0	4.31	69.0
6 × 4	$\frac{3}{8}$	12.3	3.61	3.23	51.7	3.28	52.5
5 × 3½	$\frac{5}{8}$	16.8	4.92	4.29	68.6	4.37	69.9
5 × 3½	$\frac{1}{2}$	13.6	4.00	3.50	56.0	3.56	57.0
5 × 3½	$\frac{3}{8}$	10.4	3.05	2.67	42.7	2.72	43.5
5 × 3½	$\frac{5}{16}$	8.7	2.56	2.25	36.0	2.29	36.6
5 × 3	$\frac{5}{8}$	15.7	4.61	3.98	63.7	4.06	65.0
5 × 3	$\frac{1}{2}$	12.8	3.75	3.25	52.0	3.31	53.0
5 × 3	$\frac{3}{8}$	9.8	2.86	2.48	39.7	2.53	40.5
5 × 3	$\frac{5}{16}$	8.2	2.40	2.09	33.4	2.13	34.1
4 × 4	$\frac{5}{8}$	15.7	4.61	3.98	63.7	4.06	65.0
4 × 4	$\frac{1}{2}$	12.8	3.75	3.25	52.0	3.31	53.0
4 × 4	$\frac{3}{8}$	9.8	2.86	2.48	39.7	2.53	40.5
4 × 4	$\frac{5}{16}$	8.2	2.40	2.09	33.4	2.13	34.1
4 × 3	$\frac{1}{2}$	11.1	3.25	2.75	44.0	2.81	45.0
4 × 3	$\frac{3}{8}$	8.5	2.48	2.10	33.6	2.15	34.4
4 × 3	$\frac{5}{16}$	7.2	2.09	1.78	28.5	1.82	29.1
4 × 3	$\frac{1}{4}$	5.8	1.69	1.44	23.0	1.47	23.5
3½ × 3½	$\frac{5}{8}$	13.6	3.98	3.35	53.6	3.43	54.9
3½ × 3½	$\frac{1}{2}$	11.1	3.25	2.75	44.0	2.81	45.0
3½ × 3½	$\frac{3}{8}$	8.5	2.48	2.10	33.6	2.15	34.4
3½ × 3½	$\frac{5}{16}$	7.2	2.09	1.78	28.5	1.82	29.1
3½ × 3½	$\frac{1}{4}$	5.8	1.69	1.44	23.0	1.47	23.5
3½ × 2½	$\frac{1}{2}$	9.4	2.75	2.25	36.0	2.31	37.0
3½ × 2½	$\frac{3}{8}$	7.2	2.11	1.73	27.7	1.78	28.5
3½ × 2½	$\frac{5}{16}$	6.1	1.78	1.47	23.5	1.51	24.2
3½ × 2½	$\frac{1}{4}$	4.9	1.44	1.19	19.0	1.22	19.5
3 × 3	$\frac{1}{2}$	9.4	2.75	2.25	36.0	2.31	37.0
3 × 3	$\frac{7}{16}$	8.3	2.43	1.99	31.8	2.05	32.8
3 × 3	$\frac{3}{8}$	7.2	2.11	1.73	27.7	1.78	28.5
3 × 3	$\frac{5}{16}$	6.1	1.78	1.47	23.5	1.51	24.2
3 × 3	$\frac{1}{4}$	4.9	1.44	1.19	19.0	1.22	19.5
2½ × 2½	$\frac{3}{8}$	5.9	1.73			1.40	22.4
2½ × 2½	$\frac{5}{16}$	5.0	1.47			1.20	19.2
2½ × 2½	$\frac{1}{4}$	4.1	1.19			0.97	15.5
2½ × 2	$\frac{3}{8}$	5.3	1.55			1.22	19.5
2½ × 2	$\frac{5}{16}$	4.5	1.31			1.04	16.6
2½ × 2	$\frac{1}{4}$	3.62	1.06			0.84	13.4

angles with their long legs vertical. Such a section is stiffer and more economical than other types. The size of the angles should be such that the radii of gyration about the two rectangular axes is as nearly equal as possible. Gusset plates are generally used at the joints of a truss as a part of the connections (Fig. 314). The thickness of such plates varies according to the size of the joint, together with other considerations (Art. 210). It should not be so thin as to reduce the controlling values of the rivets in bearing (Art. 27). The thicknesses of gusset plates commonly used are $\frac{1}{4}''$, $\frac{5}{16}''$, $\frac{3}{8}''$, $\frac{1}{2}''$, and $\frac{5}{8}''$. The $\frac{1}{4}''$ plates are only used for light framing, and $\frac{5}{8}''$ gussets are limited to heavy joints. The usual thicknesses are then, $\frac{5}{16}''$, $\frac{3}{8}''$, and $\frac{1}{2}''$. Under ordinary circumstances, a $\frac{3}{8}''$ average thickness may

be assumed. **For purposes of computations, the distance back to back of angles may be taken as $\frac{3}{8}''$.** The radii of gyration change slowly for small differences in this distance (Table 73). As suggested above, the least radius of gyration must be

TABLE 73*

RADII OF GYRATION FOR TWO UNEQUAL ANGLES LONG LEGS VERTICAL



Single Angle		Two Angles	Radii of Gyration, Inches					
				Axis 2-2				
Size, Inches	Wt. #/ft.	Area □″	Axis 1-1	In Contact	$\frac{1}{4}''$ Apart	$\frac{3}{8}''$ Apart	$\frac{1}{2}''$ Apart	$\frac{3}{4}''$ Apart
8 × 6 × 1	44.2	26.00	**2.49**	2.39	2.48	**2.52**	2.57	2.66
$\frac{3}{4}$	33.8	19.88	**2.53**	2.35	2.44	**2.48**	2.52	2.61
$\frac{7}{16}$	20.2	11.86	**2.57**	2.31	2.39	**2.43**	2.48	2.56
6 × 4 × 1	30.6	18.00	**1.85**	1.60	1.69	**1.74**	1.79	1.89
$\frac{11}{16}$	21.8	12.80	**1.89**	1.55	1.63	**1.68**	1.73	1.82
$\frac{3}{8}$	12.3	7.22	**1.93**	1.50	1.58	**1.62**	1.67	1.76
6 × $3\frac{1}{2}$ × 1	28.9	17.00	**1.85**	1.37	1.47	**1.51**	1.56	1.66
$\frac{11}{16}$	20.6	12.12	**1.89**	1.31	1.41	**1.45**	1.49	1.60
$\frac{5}{16}$	9.8	5.74	**1.95**	1.25	1.33	**1.37**	1.42	1.50
5 × $3\frac{1}{2}$ × $\frac{7}{8}$	22.7	13.34	**1.53**	1.42	1.51	**1.56**	1.61	1.71
$\frac{5}{16}$	8.7	5.12	**1.61**	1.33	1.41	**1.45**	1.50	1.59
5 × 3 × $\frac{13}{16}$	19.9	11.68	**1.55**	1.18	1.27	**1.32**	1.37	1.47
$\frac{5}{16}$	8.2	4.80	**1.61**	1.09	1.17	**1.22**	1.26	1.35
4 × 3 × $\frac{13}{16}$	17.1	10.06	**1.21**	1.25	1.35	**1.40**	1.45	1.55
$\frac{1}{4}$	5.8	3.38	**1.28**	1.16	1.24	**1.28**	1.33	1.43
$3\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{11}{16}$	12.5	7.30	**1.06**	1.03	1.13	**1.18**	1.23	1.33
$\frac{1}{4}$	4.9	2.88	**1.12**	0.95	1.04	**1.09**	1.13	1.23
$2\frac{1}{2}$ × 2 × $\frac{1}{2}$	6.8	4.00	**0.75**	0.84	0.94	**0.99**	1.04	1.15
$\frac{1}{4}$	3.62	2.12	**0.78**	0.80	0.89	**0.93**	0.98	1.08

RADII OF GYRATION FOR TWO EQUAL ANGLES

Single Angle		Two Angles	Radii of Gyration, Inches					
				Axis 2-2				
Size, Inches	Wt. #/ft.	Area □″	Axis 1-1	In Contact	$\frac{1}{4}''$ Apart	$\frac{3}{8}''$ Apart	$\frac{1}{2}''$ Apart	$\frac{3}{4}''$ Apart
8 × 8 × $1\frac{1}{8}$	56.9	33.46	**2.42**	3.42	3.51	**3.55**	3.60	3.69
$\frac{13}{16}$	42.0	24.68	**2.46**	3.37	3.46	**3.50**	3.55	3.64
$\frac{1}{2}$	26.4	15.50	**2.51**	3.33	3.41	**3.45**	3.50	3.59
6 × 6 × 1	37.4	22.00	**1.80**	2.50	2.68	**2.72**	2.77	2.87
$\frac{11}{16}$	26.5	15.56	**1.83**	2.54	2.63	**2.67**	2.71	2.81
$\frac{3}{8}$	14.9	8.72	**1.88**	2.49	2.58	**2.62**	2.66	2.75
4 × 4 × $\frac{13}{16}$	19.9	11.68	**1.18**	1.75	1.85	**1.89**	1.94	2.04
$\frac{1}{4}$	6.6	3.88	**1.25**	1.66	1.75	**1.79**	1.84	1.93
$3\frac{1}{2}$ × $3\frac{1}{2}$ × $\frac{13}{16}$	17.1	10.06	**1.02**	1.55	1.65	**1.70**	1.75	1.85
$\frac{1}{4}$	5.8	3.38	**1.09**	1.46	1.55	**1.59**	1.64	1.73
3 × 3 × $\frac{5}{8}$	11.5	6.72	**0.88**	1.32	1.41	**1.46**	1.51	1.61
$\frac{1}{4}$	4.9	2.88	**0.93**	1.25	1.34	**1.38**	1.43	1.53
$2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{1}{2}$	7.7	4.50	**0.74**	1.09	1.19	**1.24**	1.29	1.39
$\frac{1}{4}$	4.1	2.38	**0.77**	1.05	1.14	**1.19**	1.24	1.34

* Based upon the "Pocket Companion," Carnegie Steel Co.

established for a given section. Table 73 gives values for the common sizes of angles.† Only limiting, and one intermediate, thicknesses are given. The values change slowly so that the radii of gyration for intermediate thicknesses may be interpolated. To illustrate,

r_{2-2} for 2 ∟ 5 × $3\frac{1}{2}$ × $\frac{7}{8}$, $\frac{3}{8}''$ apart = 1.56″
r_{2-2} for 2 ∟ 5 × $3\frac{1}{2}$ × $\frac{5}{16}$, $\frac{3}{8}''$ apart = 1.45″
Difference for $\frac{9}{16}''$ = 0.11″
or 0.01+″ per $\frac{1}{16}''$.
r_{2-2} for 2 ∟ 5 × $3\frac{1}{2}$ × $\frac{1}{2}$, $\frac{3}{8}''$ apart = 1.45 + 3(0.01), say 1.49″.

For a given section, the least radius of gyration must be used. Two reference lines must be considered, namely, the 1–1 and 2–2 axes, and the smaller value noted.‡ Thus for 2 ∟ $3\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{1}{4}$, $\frac{3}{8}''$ apart, r_{1-1} = 1.12″ and r_{2-2} = 1.09″, and the value 1.09″ should be used. It will be noted that such a pair of angles is nearly ideal and economical of material, because the radii of gyration in the two directions are nearly equal. The reason why unequal legged angles are used may be made obvious by a reference to the table:

2 ∟ $3\frac{1}{2}$ × $3\frac{1}{2}$ × $\frac{1}{4}$, Weight = 2 × 5.8 = 11.6#/ft., Area = 3.88□″, r_{min} = 1.09″
2 ∟ 4 × 3 × $\frac{1}{4}$, Weight = 2 × 5.8 = 11.6#/ft., Area = 3.88□″, r_{min} = 1.28″.

Thus, for the same sectional area and weight per foot, the 4 × 3 × $\frac{1}{4}$ angles are stiffer. It may also be noted that the difference between the r_{1-1} and r_{2-2} values for equal-legged angles is considerable in all cases. By referring to a handbook of structural shapes and making similar comparisons, the reason why the common shapes shown in Table 73 are used, will be evident. For the same reason, the more common of the latter are $2\frac{1}{2}$ × 2, $3\frac{1}{2}$ × $2\frac{1}{2}$, 4 × 3, 5 × $3\frac{1}{2}$, and 6 × 4.

It should be evident that more than one pair of angles may be selected to carry a given stress. To arrive at an economical selection may mean considerable "cut and try." Some of this may be eliminated by using approximate values. Under average conditions, **a trial allowable stress of 10,000 #/□″ may be used to determine a required area as a guide.** Another aid is to solve for the minimum radius of gyration, since the length is known and the maximum ratio of slenderness is established. Thus,

$$\text{Max.} \frac{l}{r} = 120, \quad \text{or} \quad \frac{l}{120}$$

† These values are obtained by applying the ordinary principles of mechanics relative to center of gravity, moment of inertia and radius of gyration. See Table 79 also.

‡ The radius of gyration about the 3-3 axis (Art. 5) does not have to be considered for a pair of angles back to back, as each axis is in a direction opposite to the other.

In this way, small angles which have a radius of gyration less than the allowable can be eliminated. Minimum sizes are seldom the best selection, however, except for a long member with a small stress, such as in bracing. Larger legged angles of smaller thickness will usually work out to be more rigid and more economical. The minimum thickness of metal desirable will also be a factor in the selection (Art. 5). The effective or unsupported length of the compression members in steel truss work is commonly taken as the distance between the centers of the joints.*

Illustrative Prob. 206a. Design a pair of angles to carry a compression of 50,000#, if the unsupported length is 8'-0" and ⅜" gusset plates are to be used at the ends for connections. Maximum ratio of slenderness = 120. Use $p = 16{,}000 - 70\frac{l}{r}$.†

Min. $r = \frac{8 \times 12}{120} = 0.80''$. Referring to Table 73,

2½ × 2 ∟s may be eliminated.

Try 2 ∟s 3½ × 2½. r_{1-1} = 1.09" average — controls.
r_{2-2} = 1.14" average.

$$p = 16{,}000 - \frac{70 \times 8 \times 12}{1.09} = 9830\#/\square'' \text{ allowable.}$$

$$A = \frac{50{,}000}{9830} = 5.08\square'' \text{ required, or } 2.54\square'' \text{ in each } \llcorner.$$

This requires 2 ∟s 3½ × 2½ × ½‡ (wt. 18.4#/ft.).

Try 2 ∟s 4 × 3. r_{1-1} = 1.24" average — controls.
r_{2-2} = 1.30" average.

$$p = 16{,}000 - \frac{70 \times 8 \times 12}{1.24} = 10{,}590\#/\square'' \text{ allowable.}$$

$$A = \frac{50{,}000}{10{,}590} = 4.72\square'' \text{ required or } 2.36\square'' \text{ in each } \llcorner.$$

This requires 2 ∟s 4 × 3 × ⅜‡ (wt. 17.0#/ft.).

If ⅜" were the minimum desirable thickness, no further computations would be necessary. The latter pair of ∟s weighs less and would be used unless there were some special reason.

Try 2 ∟s 5 × 3 × 5/16. (Since 4 × 3 × ⅜ ∟s were satisfactory, no thicker ∟ need be tried.)

r_{1-1} = 1.61" and r_{2-2} = 1.22".

$$p = 16{,}000 - \frac{70 \times 8 \times 12}{1.22} = 10{,}490\#/\square'' \text{ allowable.}$$

$$A = \frac{50{,}000}{10{,}490} = 4.76\square'' \text{ required, or } 2.38\square'' \text{ in each } \llcorner.$$

2 ∟s 5 × 3 × 5/16 are satisfactory (wt. 16.4#/ft.).

These weigh less than the 4 × 3 × ⅜ ∟s and if 5/16" is a satisfactory thickness, they should be used for economy.

By "thumb" rules, r = 0.80" (as above).

$$\text{Trial area} = \frac{50{,}000}{10{,}000} = 5.0\square'', \text{ or } 2.5\square'' \text{ in each } \llcorner.$$

Use 2 ∟s 4 × 3 × ⅜ or
2 ∟s 5 × 3 × 5/16.

With practice and experience, the number of calculations may usually be reduced to at least two. The number of different sizes used in a truss should also be kept to a reasonable minimum. The time spent in determining which of two sections is more economical, compared with the weight saved, as in all design, is always the important commercial factor. For such reasons, tables and diagrams of the carrying capacities of different sized angles are often made, in order to minimize the time required for design.

Two angles with their short legs vertical are not commonly used for compression members. An instance where such a section may be used is when the effective length of the strut about the two axes is different. Such a case occurs for the member *de* in Fig. 315. The more efficient member has its long legs outstanding in the braced plane. The strut may bend about the 2–2 axis on a 10'-0" length, but it is restrained about the 1–1 axis by the member *cf* framing into it. The unsupported length in this direction is only 5'-0". The design is controlled by the larger value of $\frac{l}{r}$. If the lengths *df* and *ef* were not equal, the compression member should be designed for the longer dimension and the larger stress. Table 74 gives radii of gyration for common sections.

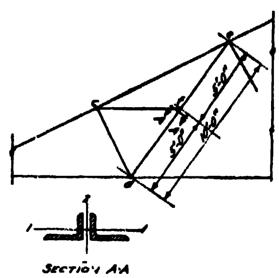


FIG. 315

Illustrative Prob. 206b. Determine a section to be used in Fig. 315 if the stress in $df = fc$ = 50,000# compression. ⅜" gusset plates.

r_{2-2} should $= 2\, r_{1-1}$ theoretically.

$$r_{2-2} \text{ (min.)} = \frac{10 \times 12}{120} = 1.0'' \quad r_{1-1} \text{ (min.)} = \frac{5 \times 12}{120} = 0.5''$$

2½ × 2 ∟s could be used as far as the radii of gyration are concerned.

$$\frac{50{,}000}{10{,}000} = 5.0\square'' \text{ trial area or } 2.5\square'' \text{ for each } \llcorner$$

No 2½ × 2 ∟s large enough.

Try 2 ∟s 3½ × 2½. r_{1-1} (average) = 0.71"
r_{2-2} (average) = 1.75"

$$\frac{5 \times 12}{0.71} = 84.5. \qquad \frac{10 \times 12}{1.75} = 68.5$$

The larger value of $\frac{l}{r}$ must be used.

$$p = 16{,}000 - 70 \times 84.5 = 10{,}080\#/\square'' \text{ allowable.}$$

$$A = \frac{50{,}000}{10{,}080} = 4.96\square'' \text{ required, or } 2.48\square'' \text{ for each } \llcorner.$$

This requires 2 ∟s 3½ × 2½ × ½ (wt. = 18.8#/ft.).

Try 2 ∟s 5 × 3. (The radii of gyration may be more carefully selected now, because a definite idea of the thickness is known.)

* Some designers make an allowance for the restraining action of the gusset plates and connections at the joints, and deduct a small distance, say 6", at each end from the length center to center of joints. This the authors do not advise.

† A number of trials will be made to illustrate how the judgment may be developed in making a selection which will be economical of weight, occupy a small space, conform to minimum thickness requirements, and provide gauge lines wide enough for suitable connections. Many texts in illustrating such work assume the best size and proceed to show that it is satisfactory, leaving the novice wondering how the assumption was made.

‡ To obtain actual areas of angles for intermediate thicknesses, Table 38 must be consulted.

$r_{1-1} = 0.85''$ $\qquad r_{2-2} = 2.47''$

$\dfrac{5 \times 12}{0.85} = 70.5 \qquad \dfrac{10 \times 12}{2.47} = 48.6$

$p = 16{,}000 - 70 \times 70.5 = 11{,}070\#/\square''.$

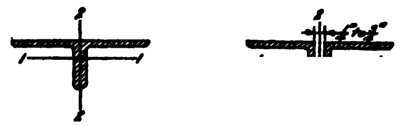
$A = \dfrac{50{,}000}{11{,}070} = 4.52\square''$ required, or $2.26\square''$ for each ∟.

This requires 2 ∟s 5 × 3 × $\frac{5}{16}$ (wt. = 16.4#/ft.).

This design could be modified, if smaller angles were desired, or a minimum thickness controlled, to 2 ∟s 4 × 3 × $\frac{3}{8}$.

TABLE 74*

RADII OF GYRATION FOR TWO UNEQUAL ANGLES SHORT LEGS VERTICAL



Single Angle		Two Angles	Radii of Gyration, Inches					
				Axis 2-2				
Size, Inches	Wt. #/ft.	Area □″	Axis 1-1	In Contact	¼″ Apart	⅜″ Apart	½″ Apart	¾″ Apart
8 × 6 × 1	44.2	26.00	**1.73**	3.64	3.73	**3.78**	3.83	3.92
¾	33.8	19.88	**1.76**	3.60	3.69	**3.73**	3.78	3.87
$\frac{7}{16}$	20.2	11.86	**1.80**	3.55	3.64	**3.68**	3.73	3.82
6 × 4 × 1	30.6	18.00	**1.09**	2.85	2.95	**2.99**	3.04	3.14
$\frac{11}{16}$	21.8	12.80	**1.13**	2.79	2.89	**2.93**	2.98	3.08
⅜	12.3	7.22	**1.17**	2.74	2.83	**2.87**	2.92	3.02
6 × 3½ × 1	28.9	17.00	**0.92**	2.92	3.02	**3.07**	3.12	3.22
$\frac{11}{16}$	20.6	12.12	**0.95**	2.87	2.96	**3.01**	3.06	3.16
$\frac{5}{16}$	9.8	5.74	**1.00**	2.81	2.90	**2.95**	3.00	3.09
5 × 3½ × ¾	22.7	13.34	**0.96**	2.36	2.45	**2.50**	2.55	2.65
$\frac{5}{16}$	8.7	5.12	**1.03**	2.26	2.35	**2.39**	2.44	2.54
5 × 3 × $\frac{13}{16}$	19.9	11.68	**0.80**	2.42	2.52	**2.57**	2.62	2.72
$\frac{5}{16}$	8.2	4.80	**0.85**	2.33	2.42	**2.47**	2.52	2.61
4 × 3 × $\frac{13}{16}$	17.1	10.06	**0.83**	1.88	1.98	**2.03**	2.08	2.18
¼	5.8	3.38	**0.89**	1.78	1.87	**1.92**	1.96	2.06
3½ × 2½ × $\frac{11}{16}$	12.5	7.30	**0.69**	1.66	1.75	**1.80**	1.86	1.96
¼	4.9	2.88	**0.74**	1.58	1.67	**1.71**	1.76	1.86
2½ × 2 × ½	6.8	4.00	**0.56**	1.15	1.25	**1.30**	1.35	1.46
¼	3.62	2.12	**0.59**	1.11	1.20	**1.25**	1.30	1.40

Occasionally, **single angle struts** may be used, particularly for light loads. The design is similar to the preceding discussion, except that the minimum radius of gyration is about the 3–3 axis. This is always less than those about the 1–1 and 2–2 axes. For this reason, single angle compression members are not economical of material. Values of the radius of gyration may be found in Table 8. Equal-legged angles are better adapted for such members, as the metal is better distributed.

Illustrative Prob. 206c. Select a single angle strut to carry a compression of 20,000# for a 6′-0″ length.

$\dfrac{6 \times 12}{120} = 0.58''$ minimum radius of gyration.

$\dfrac{20{,}000}{10{,}000} = 2.0\square''$ guide area.

From Table 8, 3 × 3 is the smallest angle which may be used ($r_{3-3} = 0.58''$).

$p = 16{,}000 - \dfrac{70 \times 6 \times 12}{0.58} = 7300\#/\square''$ allowable.

$A = \dfrac{20{,}000}{7300} = 2.74\square''$ required.

3 × 3 × ½ ∟ = 2.75□″ (wt. 9.4#/ft.).

Try 3½ × 3½. r_{3-3} (average) = 0.68″.

$p = 16{,}000 - \dfrac{70 \times 6 \times 12}{0.68} = 8700\#/\square''$ allowable.

$A = \dfrac{20{,}000}{8700} = 2.29\square''$ required.

3½ × 3½ × ⅜ ∟ = 2.48□″ (wt. 8.5#/ft.).

Try 4 × 4. r_{3-3} (average) = 0.79″.

$p = 16{,}000 - \dfrac{70 \times 6 \times 12}{0.79} = 9620\#/\square''.$

$\dfrac{20{,}000}{9620} = 2.07\square''$ required.

4 × 4 × $\frac{5}{16}$ ∟ = 2.40□″ (wt. 8.2#/ft.).

The thickness of a single angle should not be so small as to make the bearing value of the rivets at the end connection less than the value of the rivets in single shear.

Some designers take into consideration whether one or both legs are to be fastened at the end connections.† If one leg only is attached, it is reasoned that the load is eccentric to some extent and some bending is induced in the connecting rivets. It is sometimes assumed on this basis of reasoning that only the area of the attached leg should be considered as effective. Specifications in general are lax as to defining what the area of "one leg" is, but usually it is the area resulting from the product of the full width of the leg attached and its thickness. Such an extreme is not warranted.

Illustrative Prob. 206d. Select an angle for the data of Illustrative Prob. 206c, if only one leg is assumed as effective. Assume 3½″ leg is to be attached. r_{3-3} (average) = 0.68″.

$p = 16{,}000 - \dfrac{70 \times 6 \times 12}{0.68} = 8700\#/\square''$ allowable.

$A = \dfrac{20{,}000}{8700} = 2.29\square''$ required.

$\dfrac{2.29}{3.5} = 0.65''$. A 3½ × 3½ × $\frac{11}{16}$ ∟ (wt. 14.8#/ft.) is required. This section is not usually carried in stock because of its odd thickness.

Try 5 × 3½ ∟, 5″ leg attached. r_{3-3} (average) = 0.75″.

$p = 16{,}000 - \dfrac{70 \times 6 \times 12}{0.75} = 9380\#/\square''$ allowable.

$A = \dfrac{20{,}000}{9380} = 2.13\square''$ required.

$\dfrac{2.13}{5} = 0.42''$. A 5 × 3½ × $\frac{7}{16}$ ∟ (wt. 12.0#/ft.) is required.

* Based upon the "Pocket Companion," Carnegie Steel Co.

† Some authorities claim that if only one leg of an angle is attached with three or more rivets, about 80% of the full strength of the angle is developed.

Comparing the results of Illustrative Probs. 206*c* and 206*d*, it is obvious that there is no economy in attaching one leg. For this reason, **both legs are usually connected** because it is cheaper and more rigid. In practice, the designer seldom considers whether one or both legs are to be fastened, and designs as if both legs were to be attached. The assumption that only one leg is effective is too severe in the opinion of the authors.

Prob. 206e. What is the safe capacity of 2 ∟ $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$, $\frac{3}{8}$" back to back, on a 12'-0" length?

Prob. 206f. Select a pair of angles (long legs vertical) to carry a compression of 42,000# on a 10'-3" length. Use $p = 16{,}000 - 70\frac{l}{r}$ with $\frac{l}{r} = 120$ maximum. Assume $\frac{3}{8}$" gusset plates.

Prob. 206g. Select a pair of angles (long legs vertical) to carry 76,000# compression. $L = 10'\text{-}0''$.

Prob. 206h. Design a single angle strut to carry 35,000# on a 9'-0" length.

207. Design of Members Subjected to Combined Stresses.

In special instances, a truss member may have to carry a distributed or concentrated load in addition to the direct stress induced in it by the truss action. This causes bending, and creates indirect stresses. Cases of this kind occur when the top chord of a truss carries the roof supporting materials directly, when purlins intermediate between panel points are used, or when the bottom chord of a truss is subjected to distributed ceiling loads.

SPECIFICATION CLAUSE*

> Members subject to the action of both axial and bending stresses shall be proportioned so that the greatest fibre stress will not exceed the allowed limits in that member.

Since two kinds of stress are involved, the design must be carried forward by " cut and try " methods. The guide is to proportion a member somewhat in excess of that required for the axial stress and then to test it out for the combined stress, to determine whether the latter exceeds the allowable or not. A change in the section may then be made if required. Many codes make allowance for the fact that the maximum combined stress occurs only at one point along the length, and then only at one theoretical point in the cross-section.

SPECIFICATION CLAUSE

> The permissible working stress for members subjected to combined fibre stresses shall not exceed the usual allowable for axial stresses by more than 25%.

Illustrative Prob. 207a. Proportion a member to safely withstand a compression of 40,000# on a 9'-0" length, and a uniform load of 300#/ft. Use $p = 16{,}000 - 70\frac{l}{r}$ and maximum ratio of slenderness = 120. Use $\frac{3}{8}$" gusset plates.

$$\text{Minimum } r = \frac{9 \times 12}{120} = 0.9''.$$

$$\text{Trial area} = \frac{40{,}000}{10{,}000} = 4.0\square'', \text{ or } 2.0\square'' \text{ for each angle.}$$

Referring to Table 73, try 2 ∟ 4 × 3.
r_{1-1} (average) = 1.25" — controls.
r_{2-2} (average) = 1.34".

$$p = 16{,}000 - \frac{70 \times 9 \times 12}{1.25} = 9950\#/\square'' \text{ allowable.}$$

$$A = \frac{40{,}000}{9950} = 4.02\square'' \text{ required, or } 2.01\square'' \text{ for each } \llcorner.$$

Axial stress requires 2 ∟ $4 \times 3 \times \frac{5}{16}$.
For combined stress, try 2 ∟ $4 \times 3 \times \frac{1}{2}$.

$$A = 2 \times 3.25 = 6.50\square'', \quad \frac{I}{c} = 2 \times 1.9 = 3.8''^3$$

$$M = 1.2\,w \cdot L^2\dagger = 1.2 \times 300 \times (9.0)^2 = 29{,}200''\#.$$

$$s = M \div \frac{I}{c} = 29{,}200 \div 3.8 = 7{,}680\#/\square'' \text{ indirect}$$

$$p = \frac{40{,}000}{6.50} = 6{,}160\#/\square'' \text{ direct}$$

$$\text{Total} = 13{,}840\#/\square''$$

Allowable = 9950 × 1.25 = 12,420#/□".

2 ∟ $4 \times 3 \times \frac{1}{2}$ are not quite satisfactory. Probably two larger angles could be used with less weight.

Try 2 ∟ $5 \times 3\frac{1}{2}$. r_{1-1} (average) = 1.58"
r_{2-2} (average) = 1.50"— controls.

$$p = 16{,}000 - \frac{70 \times 9 \times 12}{1.50} = 10{,}950\#/\square'' \text{ allowable.}$$

$$A = \frac{40{,}000}{10{,}950} = 3.64\square'' \text{ required, or } 1.82\square'' \text{ for each } \llcorner.$$

Axial stress requires 2 ∟ $5 \times 3\frac{1}{2} \times \frac{5}{16}$.
For combined stress, try 2 ∟ $5 \times 3\frac{1}{2} \times \frac{3}{8}$.

$$A = 2 \times 3.05 = 6.10\square'' \quad \frac{I}{c} = 2 \times 2.3 = 4.6''^3$$

$$s = 29{,}200 \div 4.6 = 6{,}360\#/\square'' \text{ indirect}$$

$$p = \frac{40{,}000}{6.1} = 6{,}560\#/\square'' \text{ direct}$$

$$\text{Total} = 12{,}920\#/\square''$$

Allowable = 10,950 × 1.25 = 13,700#/□".

Use 2 ∟ $5 \times 3\frac{1}{2} \times \frac{3}{8}$.

Illustrative Prob. 207b. Design a pair of angles to carry a tension of 38,000# and a uniform load of 200#/ft. on an 8'-0" length. Maximum allowable tensile stress = 16,000#/□".

$$\frac{38{,}000}{16{,}000} = 2.13\square'' \text{ net area required.}$$

Try 2 ∟ $5 \times 3 \times \frac{5}{16}$. Gross area = 2 × 2.40 = 4.80□"
2 holes out = $2 \times \frac{7}{8} \times \frac{5}{16}$ = 0.54
Net area = 4.26□"

* Specifications for Steel Structures — American Bridge Company.

† Since a chord member is usually made continuous by more than one joint, the moment may be based upon partial continuity and $\frac{w \cdot L^2}{10}$, instead of $\frac{w \cdot L^2}{8}$, may be used.

Direct tensile stress $= \frac{38{,}000}{4.26} = 8{,}930\#/\square''$

$M = 1.2\,w \cdot L^2 = 1.2 \times 200 \times (8)^2 = 15{,}400''\#$

$\frac{I}{c} = 2 \times 1.9 = 3.8''^3$

Indirect stress $= 15{,}400 \div 3.8 = 4{,}060$

Total $= 12{,}990\#/\square''$

2 ∟s 5 × 3 × $\frac{5}{16}$ satisfactory (wt. = 16.4#/ft.)

Try 2 ∟s 4 × 3 × $\frac{5}{16}$. Gross area $= 2 \times 2.09 = 4.18\square''$

2 holes out $= 2 \times \frac{7}{8} \times \frac{5}{16} = 0.54$

$\frac{I}{c} = 2 \times 1.2 = 2.4''^3$ Net area $= 3.64\square''$

Direct tensile stress $= \frac{38{,}000}{3.64} = 10{,}470\#/\square''$

Indirect stress $= 15{,}400 \div 2.4 = 6{,}400$

Total $= 16{,}870\#/\square''$

2 ∟s 4 × 3 × $\frac{5}{16}$ unsatisfactory

2 ∟s 4 × 3 × $\frac{3}{8}$ are no gain (wt. = 17.0#/ft.)

Use 2 ∟s 5 × 3 × $\frac{5}{16}$.

The method of design is similar when a concentrated load is applied between two panel points. An occasional instance of this kind is when the roofing requires supports at a closer spacing than it is desirable to locate panel points, as illustrated in Fig. 316 (*a*). The member *mn* must be designed for the bending induced by the load *K* as well as for the direct stress in it. The moment may be based upon the component of the load perpendicular to the member. Thus in Fig. 316 (*b*), the theoretical bending is $M = \frac{P \cdot l}{4}$, and not the result obtained by using the load *K*. If the member *mn* is continuous by the joints *m* and *n*, the value of the moment may be reduced, say by the ratio $\frac{8}{10}$.* When the direct stresses in the truss members are to be obtained, the interior loads must be included at the joints. Thus in Fig. 316 (*a*), the load at joint *m* is 4000#, and so on. Similarly in Fig. 316 (*c*), the load at *p* is $3000 + 6\,(200) = 4200\#$.†

In occasional instances, some of the web members of trusses are subjected to alternate stresses due to different conditions of loading.

SPECIFICATION CLAUSE:‡

Members subject to alternate stresses of tension and compression shall be proportioned for the stress giving the largest section, but their connections shall be proportioned for the sum of the stresses.

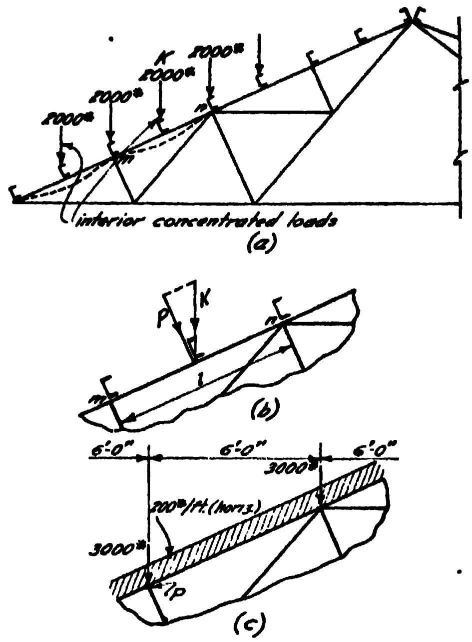


FIG. 316

Prob. 207c. Proportion a member of two angles ($\frac{3}{8}''$ back to back of angles and long legs vertical) to carry a compression of 28,000# on an 8'-0" length and a uniform load of 340#/ft.

Prob. 207d. Select a pair of angles to safely withstand a tension of 60,000# and a uniform load of 300#/ft. on a 9'-0" span.

* This is the value of the bending if the member is considered to be simply supported. If it may be assumed as partially continuous, then $M = \frac{8}{10} \times \frac{P \cdot l}{4}$, or $M = \frac{P \cdot l}{5}$, may be employed.

† Some engineers add to the direct stress obtained from the truss solution, the component of the load parallel to the member (such as the value of the component parallel to the roof in Fig. 316 (*b*)). This would be a value in pounds. Such refinement hardly seems necessary, and the authors believe such a procedure may be omitted.

‡ Specifications for Steel Structures — American Bridge Company.

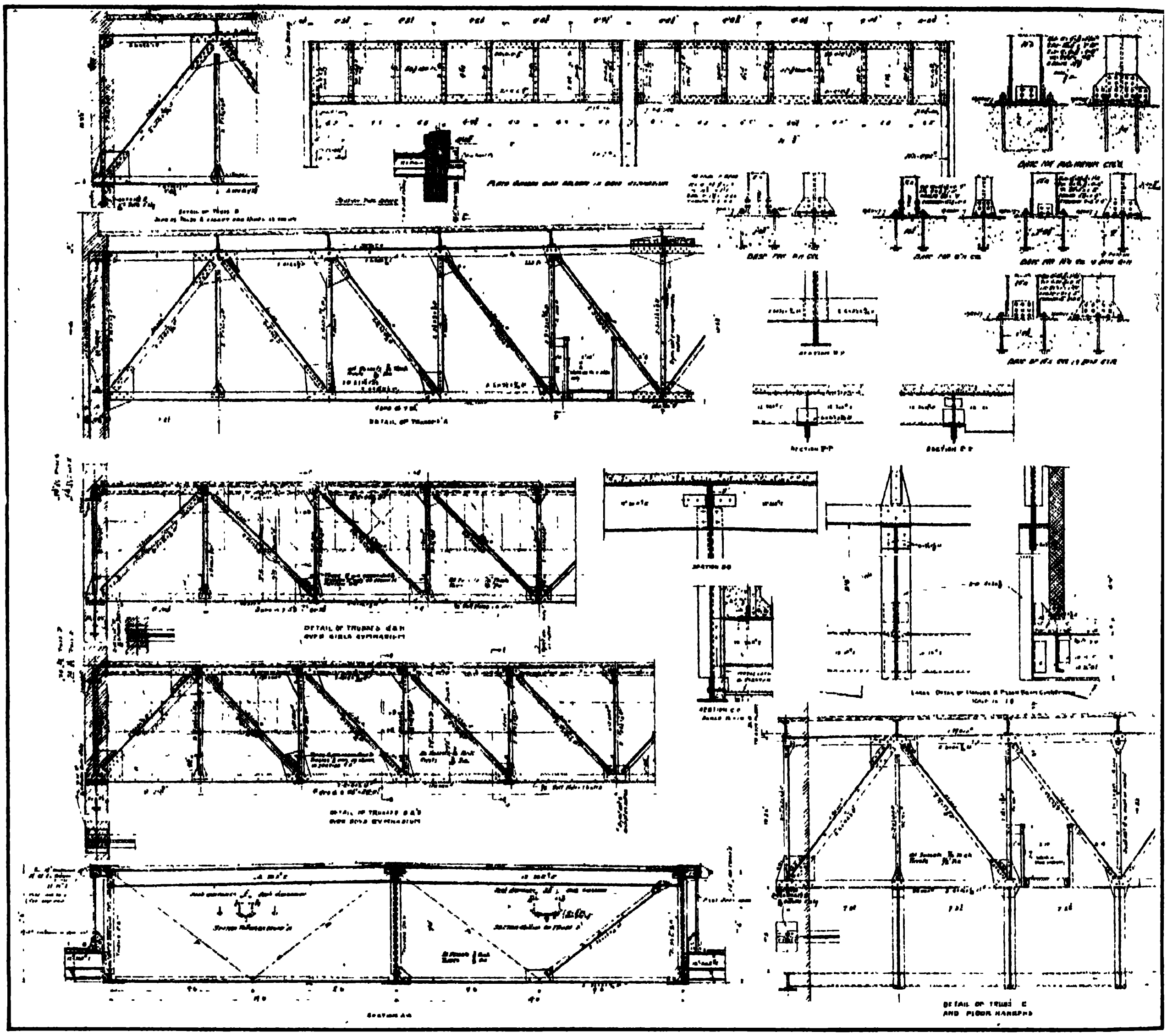


PLATE 28 THE SCHOOL
DETAILS OF GYMNASIUM GIRDERS AND TRUSSES
WALTER R. McCORNACK, ARCHITECT

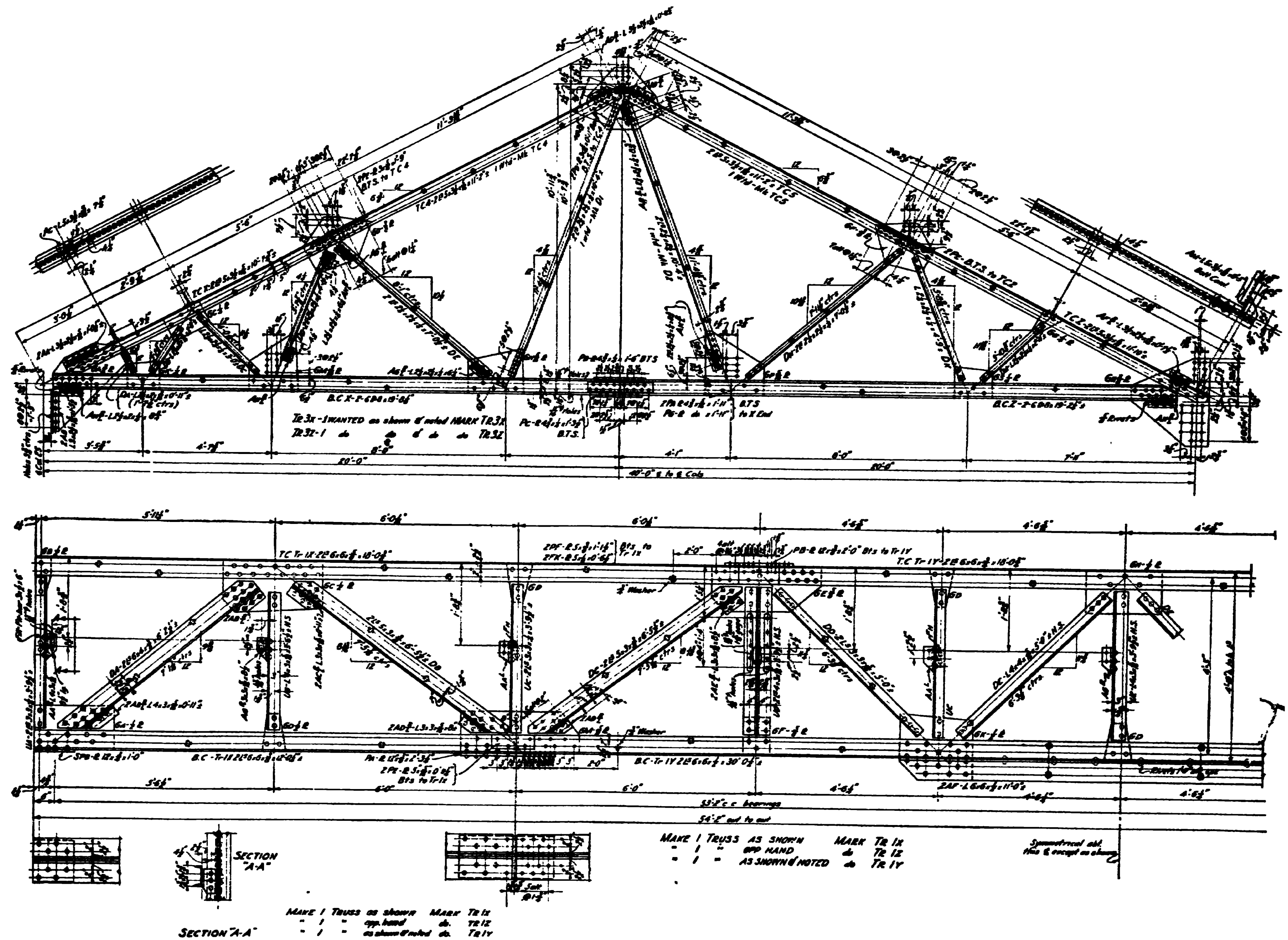


PLATE 29 TYPICAL SHOP DRAWINGS OF TRUSSES

Courtesy of The Eastern Bridge and Structural Company

CHAPTER 17

DESIGN OF TRUSS DETAILS

208. General Considerations.

The design of the details of a steel truss is generally not a part of the structural engineer's work, but is usually done at the office of the fabricating company. However, a structural designer should fully understand how such details are made, and what the important features of detail design of this kind are. He is frequently called upon to approve details submitted by structural companies. The importance of the joints in a truss should not be slighted, for a member is limited by the strength of its connections. If a designer understands structural detailing, he is much less apt to call for sizes and arrangements which will result in awkward, or perhaps impossible, details.

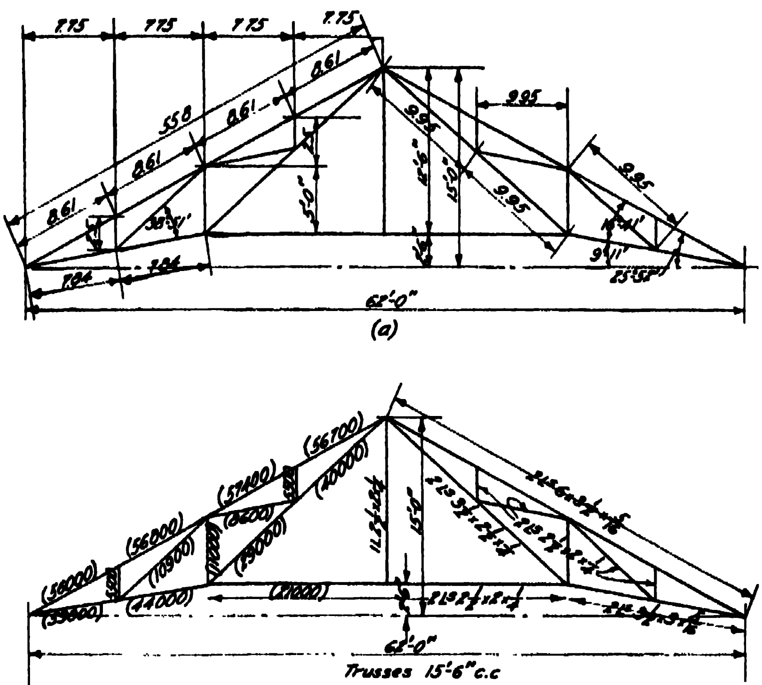


FIG. 317

One of the first steps in detailing a steel truss is to establish the **working lines.** These correspond to the dimensions and line diagrams used in the design of the truss, and for which the stresses have been established. Figure 317 (*b*) shows a typical sketch which corresponds to the data furnished by the engineer. It is necessary to determine the distances between all the **working points.** The latter are defined by the intersection of the working lines. The sketch in (*a*) represents that in (*b*) with the necessary dimensions added.

In making a truss drawing, liberties are often taken with the scales. In order to show the details at the joints, a scale of $\frac{3}{4}'' = 1'\text{-}0''$ is commonly used. If the whole truss were laid out at such a large scale the drawing would be too large in most instances. Consequently a smaller scale may be used for the distances between working points. It is of course more desirable to use one scale if possible so that a distorted idea of the truss is not obtained. Special joints may be laid out in pencil to a $1\frac{1}{2}''$ or $3'' = 1'\text{-}0''$ scale on a separate sheet, when necessary, to obtain clearances, and the like, and the scaled dimensions indicated on the truss drawing.

On a theoretical basis, the center of gravity axes of all the members ought to be made to coincide with the working lines. This is impractical for a number of reasons. The important one is that working lines should intersect in a common point at any joint, and that the rivets should be driven on these lines in order to avoid eccentricity of one member with reference to another. The center of gravity line of an angle does not correspond to the standard gauge line. In practically all instances, the center of gravity line is too near the back of the angle to allow rivets to be driven on it (Art. 5). For this reason, **the gauge lines are made coincident with the working lines.** If an angle leg permits the use of two standard gauge lines, the one nearer the center of gravity axis would naturally be used. This is always the gauge line nearer the back of the angle. This is illustrated in Fig. 318 (*b*). A good argument for using gauge lines as working lines is that the rivets are to be driven on such gauge lines, and

as the rivets really take the stress into a joint, they should be on the working lines. If center of gravity axes were used, extra labor and confusion would result in making drawings and templates, and in the fabrication.

As suggested above, some eccentricity is probably introduced by such practice. Thus in Fig. 318 (a), the center of gravity axis of a $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ ∟ is 1.01" from the back, while the gauge line is out a distance of $2\frac{1}{4}$". If the stress were assumed to be acting on the center of gravity axis, it would be 1.24" eccentric about the joint. In practice, any moment generated by such eccentricity is neglected, and the number of rivets used is made sufficient to provide for this variation by conservative design. Where several members frame into a joint, the moment induced by one member may be offset by that caused by another.

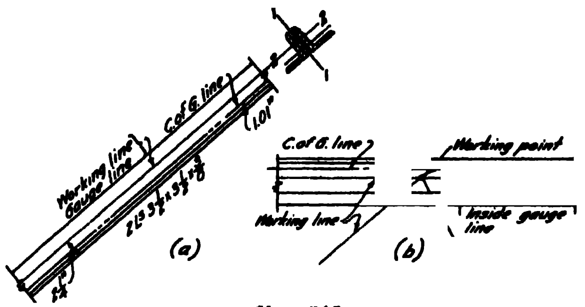


FIG. 318

A detail which must be established is the relation of the outstanding legs of the angles to the working lines. The angles constituting the top and bottom chords naturally have their outstanding legs placed above and below the working lines, respectively, as illustrated in Fig. 319 (a). For web members, there is a difference in practice in this regard. If the outstanding legs are turned upward, as in (b), dirt will not collect at the lower ends of the members so readily. There is also less chance for corrosion in trusses exposed to the weather, as water will not be retained as much. If the **outstanding legs** are **turned downward,** as in (c), they make a better appearance, especially when looking up from under the truss. The latter method is recommended, as the joints may also be made more compact in many instances.

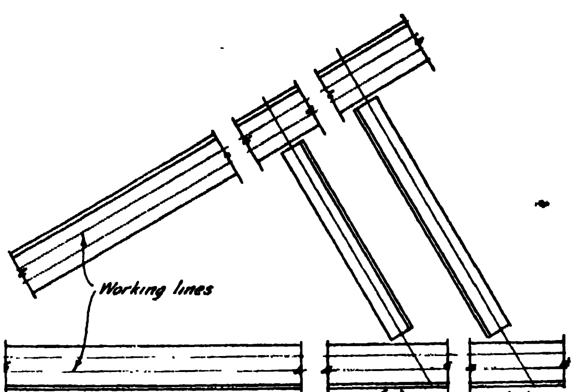


FIG. 319

When a member terminates at a joint, as in Fig. 320 (a), the full stress must be developed by the rivets. If a member is continuous by a joint, as in (b), only the difference in the adjacent stresses needs to be resisted by riveting. In many cases, the stress to be developed at the end of a member may be large, so that a considerable number of rivets is required. If all of these were driven through the legs of the angles adjacent to the gusset plate, a large plate would be required. In order to avoid this, a **clip angle** may be used, as shown in Fig. 321 (b). A considerable amount of plate may be saved, as is evident by comparing (b) with (a). Extremely large gusset plates mar the appearance of a truss, and the joint is not compact. Another important purpose of the clip angle, however, is to develop the strength of the other leg of the main angle. This should always be done for large stresses, as the load is more uniformly distributed throughout the gusset plate. A rule of thumb which may be used is that **when the required number of rivets exceeds five in a single gauged angle, a clip angle should be employed.** Judgment as to the appearance of the joint is required, of course, as the details are developed.

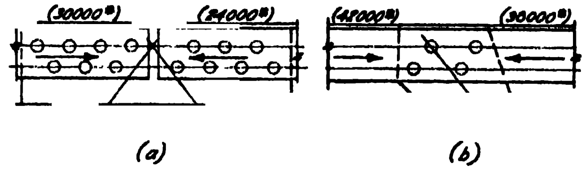


FIG. 320

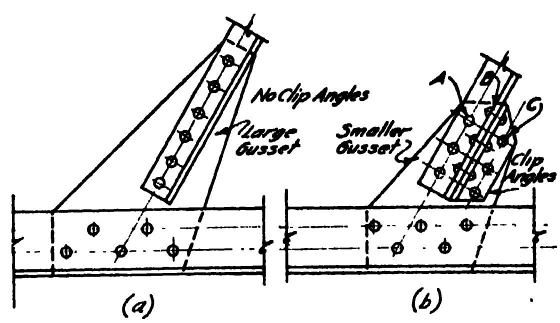


FIG. 321

The size of the clip angles is dependent to some extent upon the main angles. The outstanding legs of both should be of the same width so that the gauge lines will match. The width of the legs of the clip angles, adjacent to the gusset plate, should be great

enough to allow the driving of rivets in them, and it must of course conform to a stock size of angle. The same number of rivets should be driven in each of the outstanding legs of the clip angles as are driven in the legs against the gusset, in order to realize the full strength of the clip angle.* They are usually made to stagger with the rivets in the plane of the gusset. Of the number of rivets to place in the clip angle, the number of rivets required to develop the stress in a member depends upon the arrangement of a given joint. The aim is to keep the gusset plate small, the group of rivets compact, and to develop a "balance." The last expression means that the moments of the rivets on each side of the center of gravity of the member should theoretically be equal. This is illustrated by a reference to Fig. 322. Assigning a rivet value

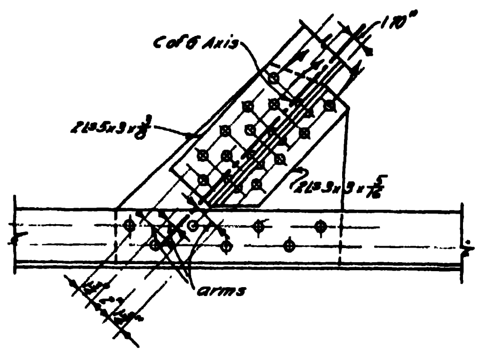


FIG. 322

of 1#, the moment of the clip angle rivets is 3 (1.75 + 1.70) = 10.45″#, and the moment of the rivets in the member is 4 (2.00 − 1.70) + 4 (3.75 − 1.70) = 9.40″#. This is a reasonably good balance. Of course such calculations would not usually be made in practice, but the spirit of balance should be carried out by inspection at least. A guide is to apportion the number of rivets according to the respective metal in the legs of the member.

209. Field Joints.

In detailing a truss, a maximum number of joints should be made with shop rivets in order to increase the efficiency and decrease the cost (Art. 27). Circumstances will require, however, that a certain number of connections be made in the field (field riveted joints). One consideration is that of the shipment of the truss from the fabricating shop to the building site. If the trusses are to be shipped by rail, the question of **clearances in transit** under bridges and in tunnels must be considered. This naturally varies for different routes, but a height above the surface of a flat car of 11′-0″ should be the maximum unless something more definite is established for a given case. The depth of a shipping piece may be made somewhat in excess of this figure, as it may be loaded as illustrated in Fig. 323. This should not exceed a dimension of say 12′-6″. Many fabricating concerns develop tables of routing clearances. The length of the flat cars (usually 40′-0″) is also an influencing factor, although more than one car may be used in train to carry a single fabricated unit. Similar data are necessary when shipment is to be made by auto trucks. When trusses are to be shipped by water, information relative to the dimensions of hatchways must be obtained. In special instances, trusses may be shipped "knocked down," that is, with each member a separate piece. This should be avoided when possible, as field riveted joints are necessarily larger and more expensive, on account of the lesser values usually allowed for field rivets and the increased cost of driving rivets on the job.

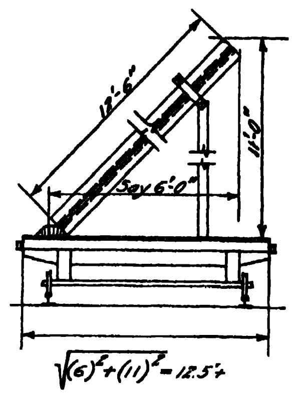


FIG. 323

Other features which should be considered in the selection of field joints are the method of erection† and the capacity of the erection equipment. In some cases, the separate fabricated units of a truss are assembled on the ground at the building site and then hoisted into position as a whole. It is more common to hoist the parts into position, however, support them by falsework, and then to rivet them into place. If the capacity of a derrick were 10 tons and a truss weighed 25 tons, it would either have to be hoisted in sections, or more than one derrick would have to be used.

The object of such study as the above is to decide which joints must be field riveted. For ordinary trusses, the joint at the peak of the truss is a point of field connection, as well as two points in the bottom chord, preferably symmetrical about the center-line. This is illustrated in Fig. 324 (*a*). The member, C1, is made separate. There are in this case three shipping pieces, Tr1x, Tr1y, and C1. The peak gusset is usually riveted to Tr1x. Figure 324 (*b*) and (*c*) shows instances for other types of trusses.

* The object of the gusset plate is to transfer the stress in a minor member to a major member. Thus the rivets at *A* in Fig. 321 (*b*) take stress directly into the gusset. For those at *C* to be thrown into action, the proportionate part of the stress in the angles goes through the rivets at *B*, through the clip angles, and thence to the rivets at *C*.

† In very special cases, the erection of other materials ahead of that of the trusses may influence the number of field joints. In alteration work, this is especially so. Some particular connections might have to be field bolted. Turned bolts are usually more expensive than rough bolts for such work.

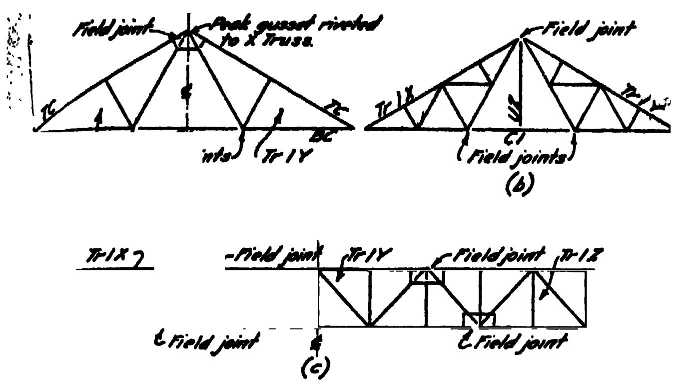


FIG. 324

210. Gusset Plates.

Where several members of a truss come together at a joint, it is necessary to provide a gusset plate which will transfer the stresses from one member to another. Figure 325 shows some typical instances. Its dimensions must be such that it will accommodate the required number of rivets at a joint and provide sufficient edge distance for all the rivets passing through the plate. These dimensions are

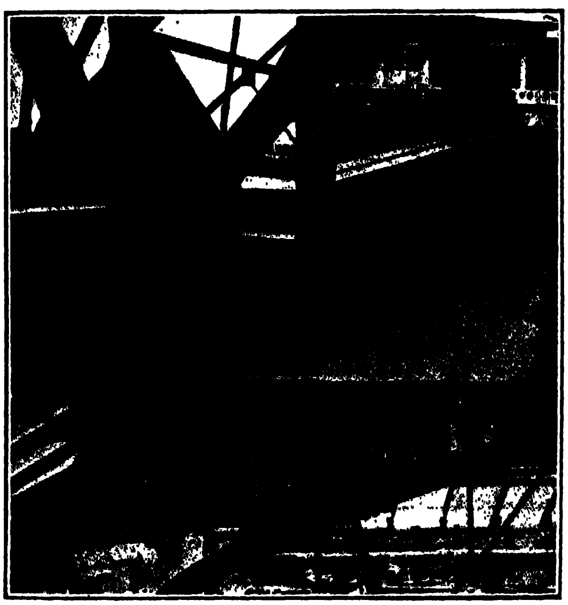


FIG. 325

seldom actually established as fixed figures, but are more frequently left as a part of the template maker's work, in which he must enclose the rivets as shown, and provide proper edge distances for them. The plates and rivet spacings are shown to scale on the details. For purposes of estimating the weight of the gussets, the dimensions of a rectangular plate from which the plate may be cut are used. When plates are ordered or reserved for gussets, the width may be shown on the drawing. Some fabricators, in their details, do not space the shop rivets with actual dimensions, but simply indicate the number required and the arrangement to scale. When the template is made, the rivets are accurately located by the template maker, using the rules for riveting dimensions for the spacings center to center, and for the edge distances. In other shops, every rivet is definitely tied into a working point by dimensions. All open holes, however, must be positively defined in any event. The authors believe that the exact location of all shop rivets is an unnecessary refinement and that this work may be done in the template shop.

The thickness of the gusset plates, however, must be established by the detailer. These are more or less fixed by rules of practice. There are two factors which influence the selection of thicknesses. One is to obtain as small a plate as possible and of a minimum weight, and the other is to develop the maximum efficiency of the rivets. If a thick plate is used, the bearing value of the rivet is increased so that fewer rivets are needed, and consequently a smaller plate may be used. Nothing is gained of course in using a thickness greater than that which develops the double shearing strength of the rivet, except to provide extra shearing, or tensile, section area in the plate itself. The thickness never needs to be greater than $\frac{1}{8}''$ less than the diameter of the rivet, to develop the full resistance of the rivet to double shear. The thickness also need never exceed the sum of the thicknesses of the two angles as no gain would result in bearing resistance (Art. 27). It is not always economical to use plates as thick as the above requirements, however. While a thinner plate may provide less bearing value for the rivets and hence have to be made larger to accommodate the increased number, it may weigh less and be cheaper. As stated before, gusset plates which are too large tend to mar the appearance of a truss, however.

As a result of such study, common thicknesses of gusset plates have been established. **The usual values employed are $\frac{5}{16}''$ and $\frac{3}{8}''$ for intermediate joints, and $\frac{3}{8}''$ and $\frac{1}{2}''$ for such connections as at the heel, peak, and splices.** The thickness to be used is a matter of judgment, depending upon the relative sizes of the members, but it is advisable to keep the number of thicknesses to a minimum. A thickness of $\frac{5}{8}''$ is employed when the size of the plate becomes too great, or when area is needed for tension or shear in the plate itself. Specifications for minimum thicknesses of metal may also limit the values. Gussets for lateral bracing and small trusses may be made $\frac{1}{4}''$ or $\frac{5}{16}''$ thick. From a

theoretical standpoint, the thickness in all cases must be sufficient to resist the shearing and tensile stresses set up in the plate by the forces which the members connected exert.* **Under ordinary circumstances, gusset plates are amply strong in shear and tension if the above proportions are followed.**

The shape of the gusset plate should be made as compact, trim, and square as possible (Fig. 326).

211. Intermediate Joints.

Figure 327 shows several typical intermediate joints in ordinary types of steel trusses. **The members are usually cut off square** at their ends except in special details. Thus the angle in (*c*) is cut on the line AB rather than on BC, even though the gusset plate has to be made slightly larger. A **clearance,** as at D, should be allowed. This is made about $\frac{1}{2}''$. The distance, E, is scaled from the truss drawing at each end of the member.

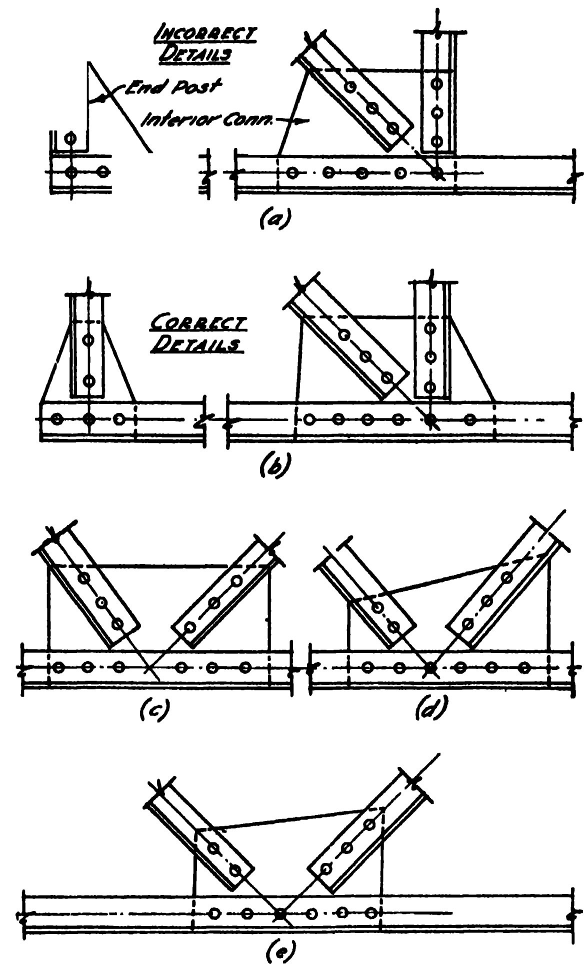


Fig. 326

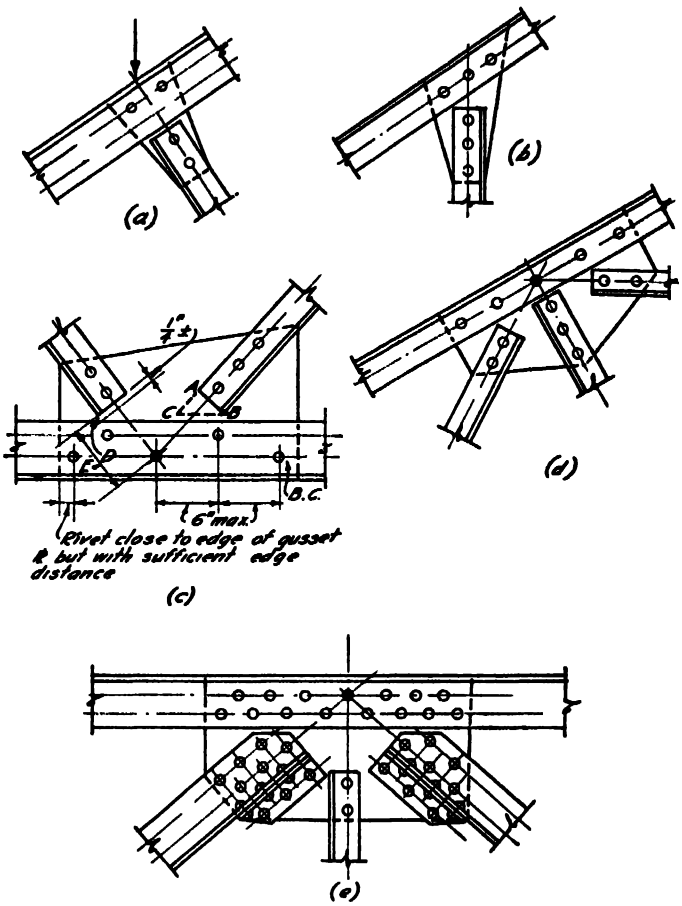


Fig. 327

The plates should not be cut flush with the backs of web members, as shown in Fig. 326 (*a*), but should be as in (*b*). Square plates should be used where possible, as in (*c*). When the rivets are unbalanced as in (*d*), a square plate cannot be used. The corners of all plates should come under the edges of the angles, as shown, rather than like the detail (*e*).†

* See articles in Engineering News Record, August 5, 1920, pp. 241-242 and p. 250.

† In foreign work the custom was to cut gussets to arcs of curves tangent to the members joined. Labor was cheap and material high, which explains this practice. This sort of fabrication also produced much more sightly steelwork, when left exposed.

The sum of these distances is then subtracted from the distance between working points to establish the length of the angles. The latter is given to the nearest $\frac{1}{8}''$ and labeled $\pm$.

No member should be connected with less than two rivets. If only one were used, there would be a tendency of the member to rotate. The second rivet overcomes this, and also provides protection in case the other rivet is poorly driven. When only two or three rivets are required to develop the difference of the stresses either side of a joint in a continuous member, as in Fig. 327 (*c*), enough

rivets should be used in practice to fill out the gusset. They should not exceed a 6″ pitch, and one should be driven near the edge of the gusset plate to supply stiffness. The latter distance is usually made the standard edge distance. Enough rivets should be used at any point to develop a local load. Thus those in Fig. 327 (*a*) should be strong enough to develop the reaction of the purlin, as well as develop the difference of the stresses each side of the joint. In like manner, the number of rivets in the top or bottom chords of a truss should always be at least equal to the maximum number in any member framing into the gusset. The required number of rivets at the end of any member is found by dividing the stress by the controlling value of the rivets (Art. 27)

Illustrative Prob. 211a. Determine the required number of rivets for the joint in Fig. 328 (*a*). Use ¾″ rivets and a ⅜″ gusset plate. Maximum shear, shop rivets, 12,000#/□″, bearing 24,000#/□″.*

Double shear, ¾″ rivet = 10,600# (Table 24).

Bearing ⅜″ plate = 6,750# (Controls).

Bearing on 2-¼″ ∟s = 9,000#

Members AD and AE $\dfrac{7250}{6750} = 1+$. Use 2 rivets.

Member AC $\dfrac{16,300}{6750} = 2+$. Use 3 rivets.

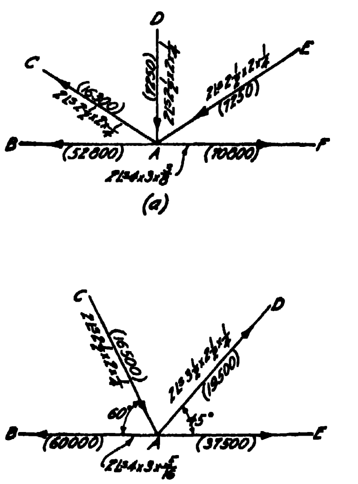


FIG. 328

3 rivets may be used in AE, opposite AC, in order to keep a balanced gusset plate.

Stress in AF = 70,800
Stress in AB = 52,800
Difference = 18,000

$\dfrac{18,000}{6750} = 2+$ 3 rivets required.

* Some designers use a value of 30,000#/□″ for the enclosed bearing on the gusset plate when it is confined between two angles (Art. 27).

More than 3 may be used on account of the size of the gusset plate.

The investigation for the tensile strength of the gusset plate may also be made.

Stress difference = 18,000

Net area required = $\dfrac{18,000}{16,000} = 1.13$□″

$\dfrac{1.13}{0.375} = 3.0″$. Reduction diameter for ¾″ rivets = ¾ + ⅛ = ⅞″. There are 2 rivets in AD.

Height of gusset required = 3.0 + 2 × ⅞ = 4¾″. This height will more than be supplied in an average detail, so this requirement is satisfied. A critical case occurs when a member carrying a relatively large stress terminates at a joint.

Stitch rivets should always be provided in double-angled members between the joints, in order to make the two angles act together. These are illustrated in Fig. 329 (*a*). Theoretically none are needed in tension members but they are generally used. For compression members, the spacing be-

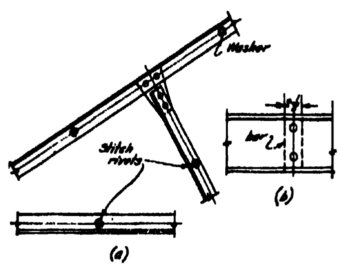


FIG. 329

tween them should be such that the ratio of slenderness for one angle, based upon this distance, is not less than the ratio of slenderness of the member as a whole. Under ordinary circumstances, the spacing so calculated is a large value and a smaller distance is used. In practice, **rivets are placed not more than 2′-0″ on centers in compression members, nor more than 3′-0″ on centers in tension members.** A washer is placed between the angles to fill the space between them. For double channel members, a bar may be used, as indicated in Fig. 329 (*b*).

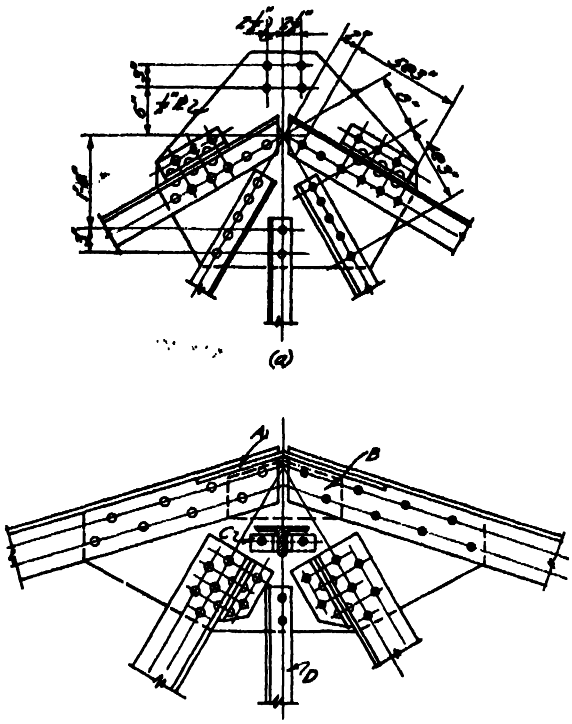
Prob. 211b. Determine the required number of ¾″ shop rivets for the joint shown in Fig. 328 (*b*). Use a $\frac{7}{16}$″ gusset plate. Maximum shear for shop rivets 10,000#/□″, bearing 20,000#/□″. Make a detail of the joint at a scale of ¾″ = 1′-0″.

Prob. 211c. If the stress in member AD in Prob. 211 (*b*) were 31,000#, and that in AC were 24,000#, make a detail of the joint.

212. Peak Joints.

The connection at the top of a truss at the center line of the span is often called a peak joint. In the usual case, it is a field connection (Art. 209). Here the controlling value of field rivets must be used for

the members that are to be attached later. It is customary to make the details of the joint symmetrical about its center line increasing the number of shop rivets to equal the number of field rivets required in the corresponding members (Art. 27). Figure 330 shows two typical details. In some instances, the gusset plate is extended above the top chord to serve as a connection for the purlin, as in (*a*). Batten plates, such as *A* in Fig. 330 (*b*),



(*a*)

(*b*)

Fig. 330

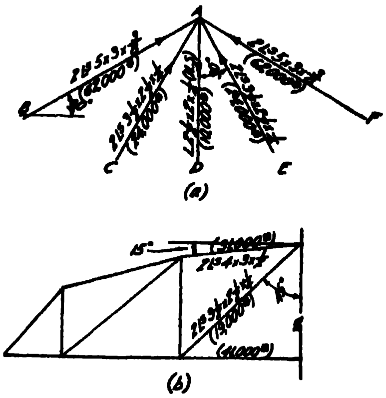
are occasionally used. These have no particular value in transferring stress but they help to stiffen the joint. Instead, flange batten plates, such as *B*, may be used. These are more efficient than the former. If a member, such as *D*, occurs at the center line to act as a sag-tie, it is connected to the peak gusset near the bottom as shown in (*b*). This allows room for a ridge strut connection, *C*, and saves some length in the angles, *D*. The required number of rivets is determined as previously discussed (Art. 211).

Illustrative Prob. 212a. Determine the required number of rivets for the joint in Fig. 331 (*a*). Use ¾" rivets and a ½" gusset plate. Maximum shear, shop rivets = 12,000#/□", bearing = 24,000#/□". Maximum shear, field rivets = 10,000#/□", bearing = 20,000#/□".

Make details symmetrical about center line of truss, use field rivet values.

Double shear, ¾" field rivets = 8840#.
Bearing ½" plate = 7500#

Member AF, $\frac{62,000}{7500} = 8+$. Use 9 rivets.
Use same in AB.
Member AE, $\frac{24,000}{7500} = 3+$. Use 4 rivets.
Use same in AC.
Single shear, ¾" field rivets = 4420#
Bearing on ¼" L = 3750#
Member AD, $\frac{10,000}{3750} = 2+$. Use 3 rivets.



(*a*)

(*b*)

Fig. 331

Prob. 212b. Design the peak joint in Fig. 331 (*b*). Use the same working stresses as in Illustrative Prob. 212*a*.

Prob. 212c. Make a detail at a scale of ¾" = 1'-0", for the data in Illustrative Prob. 212*a*.

213. Heel Joints.

The detail where the top and bottom chords of a sloped, top-chord truss come together is often called the heel joint. Several alternate details for this connection are shown in Fig. 332, for masonry wall bearing. The design of these joints is similar to that of the others except for the special conditions surrounding the bearing of the truss on its support. The gusset plate should have ample net section to resist the shearing and tensile forces exerted upon it.* The rivets immediately over the bearing plate must be strong enough to transfer the vertical component of the top chord stress and the end purlin load to the support. This force is of course equivalent to the maximum end reaction. For this reason, it is desirable to keep the rivets in the bottom chord connection symmetrical about the center-line of the bearing. To do so, the dimension *a* in Fig. 332 (*b*) usually becomes larger than the available length of bearing provided by the wall. The detail shown in (*b*) has this disadvantage. In this type of joint, the gusset is extended above the top chord so that clip angles may be used for this member. These help to make the resistance more nearly

* The end of a truss should have a depth of at least 6" over the center line of the bearing to provide a reasonable section.

coincident with the center of gravity axis of the top chord. The outstanding legs of the bottom chord, however, are not developed by this detail. This connection is satisfactory if the wall continues up by it, so that the roofing does not come down to an eave at this point.

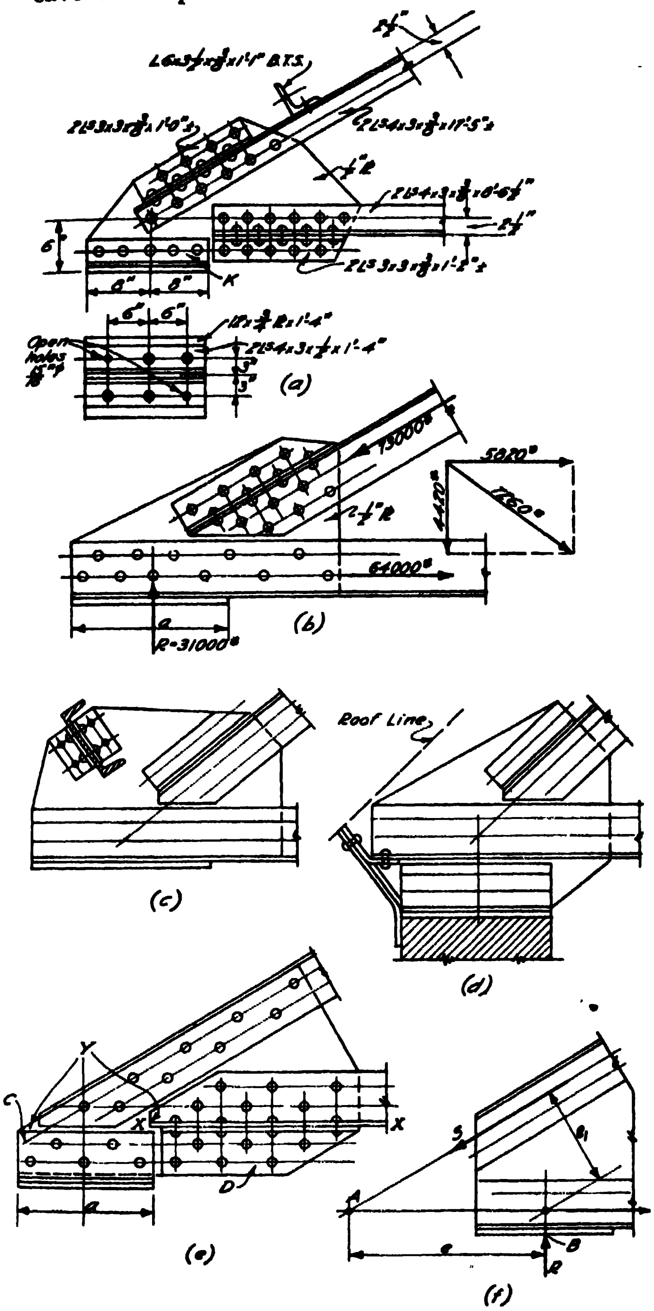


FIG. 332

Figure 332 (*a*) shows an alternate detail which may be used when the bearing length is limited. This is a good detail, as clip angles are used on both the top and bottom chord connections, thereby reducing the size of the gusset plate. When it is desirable to have a purlin connection near the end of the truss, the gusset may be extended, as indicated in (*c*). The detail in (*d*) allows a special support for the cornice construction. One object of extending the gusset plate above the top chord and below the bottom chord is to avoid concentrating the stresses from the members at the upper and lower edges of the gusset plate.

Figure 332 (*e*) shows a detail in which the gusset plate does not extend above the top chord, thereby allowing the end purlin to be placed where desired. The angles, *C*, are used so that the top chord may be stopped, as the latter, if brought down to the support, would lack bearing area. The rivets in the angles, *C*, must develop the end reaction. With a fixed value of *a*, a single or double gauged angle will be needed according to the required number of rivets. The angles, *D*, are usually made of the same size. This detail allows good lateral bracing connections at the plane *X–X*. In all details, angles should never be brought to a "feather edge," but should be cut as shown at *Y*.

Another alternate detail is shown in Fig. 332 (*f*). The working point is at *A* while the center of bearing is at *B*. This introduces eccentricity, which is not desirable, because **secondary stresses** are developed in the joint. Such a joint may be avoided when the truss rests upon a wall, but is quite common when the truss frames into a column. The distance, *e*, should be kept as small as possible, as the bending moment induced varies directly with it. It is often made so that a $\frac{1}{2}''$ clearance is maintained between the top and bottom chord angles. It should, however, be established so that the distance center to center of bearings is not an odd dimension. The first panel length must be adjusted to conform with the other dimensions. The total number of rivets must be increased from the usual required number to provide for the indirect, as well as the direct, stresses. The following relations may be used for this purpose:

$$n^2 \cdot r - R \cdot n = \frac{6\,R \cdot e}{p} \text{ (Bottom Chord), and}$$

$$n^2 \cdot r - S \cdot n = \frac{6\,S \cdot e_1}{p} \text{ (Top Chord), in which}$$

n = the total number of rivets required in the connection of each member, respectively,
r = the maximum allowable value for the rivets,
R = the maximum end reaction,
p = the pitch of the rivets, in inches, and
S = the stress in the top chord.

These equations are quadratics and may be solved by "completing the square," to obtain n in each case. A common value for the pitch of the rivets, p, is 3″.

Illustrative Prob. 213a. Determine the necessary number of rivets for the joint shown in Fig. 332 (b). Use 10,000#/□″ shear and 20,000#/□″ bearing for ¾″ rivets, and a ½″ gusset plate.

Top chord, stress = 73,000#
Double shear, ¾″ shop rivet = 8840#
Bearing on ½″ plate = 7500#

$\frac{73,000}{7500} = 9+$. Use 10 rivets as shown.

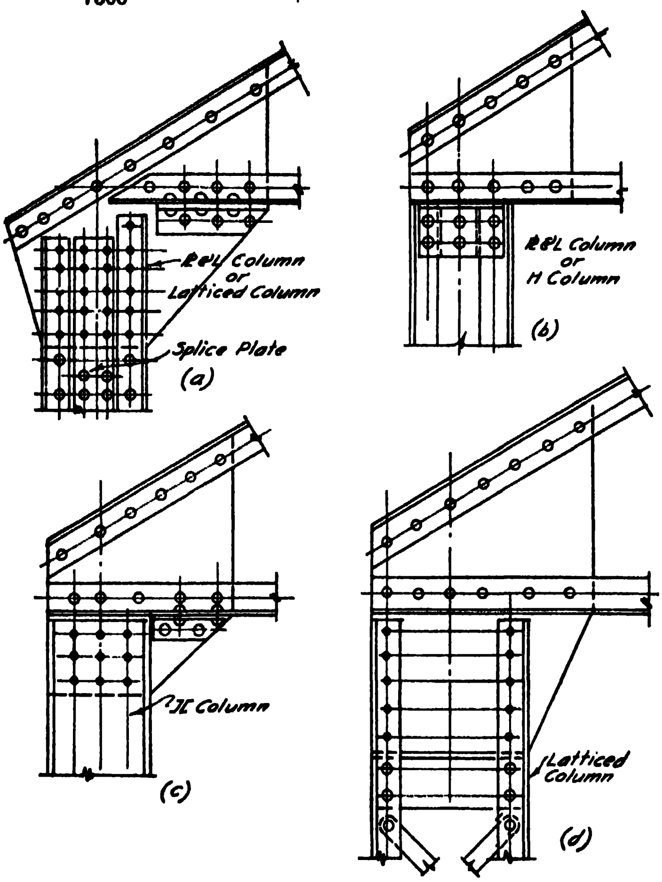


FIG. 333

Bottom chord, stress = 64,000#.

$\frac{64,000}{7500} = 8+$. Try 11.

Force due to BC stress $= \frac{64,000}{11} = 5820\#$ (horizontal) (on each rivet).

7 rivets over bearing plate.
Reaction = 31,000#.

$\frac{31,000}{7} = 4420\#$ (on each rivet over bearing).

Combined stress, as illustrated = 7260#
Allowable = 7500#

Use 11 rivets as shown.

The above computations are a matter of "cut and try" to arrive at a correct number. In practice, calculations are made to show that a selected number of rivets is satisfactory.

Illustrative Prob. 213b. Are the rivets in the angles "K" in Fig. 332 (a) sufficient if the end reaction is 35,000#? Use ¾″ rivets and the allowable values in the above illustration.

Controlling values of rivets = 7500# (see above).

$$\left.\begin{array}{r} 5 \times 7500 = 37,500\# \\ \text{End reaction} = 35,000\# \end{array}\right\} \text{O.K.}$$

Illustrative Prob. 213c. Determine how many rivets are required in Fig. 332 (f) if $S = 50,000\#$, $R = 25,000\#$, $T = 43,000\#$, $e = 14''$, and $e_1 = 7\frac{1}{2}''$. Use ⅞″ shop rivets, ⅝″ gusset plate, 12,000#/□″ shear, and 24,000#/□″ bearing. Assume 3″ pitch of rivets.

Double shear, ⅞″ shop rivet = 14,430#
Bearing, ⅝″ plate = 13,130# (Controls)

Bottom Chord

$$n^2 \cdot r - R \cdot n = \frac{6\,R \cdot e}{P}$$

$$n^2\,(13,130) - 25,000\,(n) = \frac{6 \times 25,000 \times 14}{3}$$

$$13,130\,n^2 - 25,000\,n = 700,000$$
$$n^2 - 1.9\,n = 53.2$$
$$n^2 - 1.9\,n + (0.95)^2 = 53.2 + (0.95)^2$$
$$(n - 0.95)^2 = 54.1$$
$$n - 0.95 = \pm\sqrt{54.1} = \pm 7.4$$
$$n = 7.4 + 0.95 = 8.35 \quad \text{or} \quad 9 \text{ rivets.}$$

Top Chord

$$n^2 \cdot r - S \cdot n = \frac{6\,S \cdot e_1}{P}$$

$$n^2\,(13,130) - 50,000\,n = \frac{6 \times 50,000 \times 7.5}{3}$$

$$n^2 - 3.8\,n = 57$$
$$n^2 - 3.8\,n + (1.9)^2 = 57$$
$$n - 1.9 = \pm 7.8$$
$$n = 7.8 + 1.9 = 9.7 \quad \text{or} \quad 10 \text{ rivets.}$$

When trusses frame into columns, details similar to those shown in Fig. 333 may be used. The same principles of design apply as previously explained. The exact details must, of course, conform to the type and size of the supporting columns. When the column is continuous by the end of the truss, details similar to those discussed for the supports of plate girders (Art. 82) under similar circumstances may be used. The trusses in such cases are often parallel-chorded frames, so that end connection angles, or standard seat angle and stiffener details, may be used. Similar details may be employed for trusses with sloped chords. Figure 334 illustrates the general nature of such framing. The design of end connection angles is similar to that for beam connections (Art. 29). That of seat angles becomes a part of the column detail design (see Index).

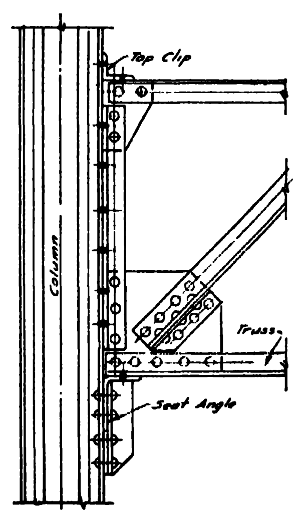


FIG. 334

Prob. 213d. Determine the required number of rivets for the heel joint in Fig. 335 (*a*). Use 12,000#/□″ shear, and 24,000#/□″ bearing for ¾″ rivets and a ⅜″ gusset plate. Use a detail similar to Fig. 333 (*a*) and make a sketch of the joint at a scale of 1″ = 1′-0″.

Prob. 213e. Repeat Prob. 213*d* for the heel joint shown in Fig. 335 (*b*).

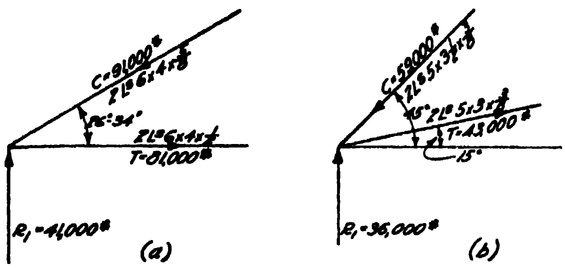


Fig. 335

214. Splices.

In the usual truss, the lengths of the chord members are such that intermediate splices are unnecessary. The stresses in successive panels vary. Economy dictates whether to make a chord member one size over its full length and omit splices, or to change the size at some intermediate panel point and develop the full stresses at each side of the joint. Under ordinary circumstances, the material which might be saved by the latter is offset by the extra fabrication required, so that it is customary to use one size of chord member from the end of the truss to a field connection. **Thus the necessary splicing usually occurs at the field joints only.**

Figure 336 shows details for such splices. The connection should be located so that a gusset plate may be used for the splice. It should have sufficient net section to resist the maximum stress acting in it. The number of rivets on each side of the splice line must be sufficient to develop the respective stresses in the members. A bottom batten plate, such as *A* in Fig. 336 (*a*), should be used, as it helps to stiffen the splice. Its more important function, however, is to develop the outstanding legs of the angles spliced. In this way, fewer rivets need be driven in the legs of the angles adjacent to the gusset plate, and this plate may be made smaller. The batten plates take the place of clip angles, which, if used, would produce awkward details for chord connections. The pair of holes adjacent to the splice line on the shop riveted side, as "*B*" in Fig. 336 (*b*), is generally left open to allow adjustment during erection. A batten plate is not of much use, however, when the members to be spliced change direction, as illustrated in (*c*). In some instances, small, vertical, batten plates are used in addition, as shown in (*b*) and (*c*). These must be "bolted to ship" (B.T.S.), as they are attached after the other rivets are driven. They provide additional lateral resistance to the splice. In all cases, **a plan view of the splice should always be shown,** as illustrated, in order to fully locate all open holes.

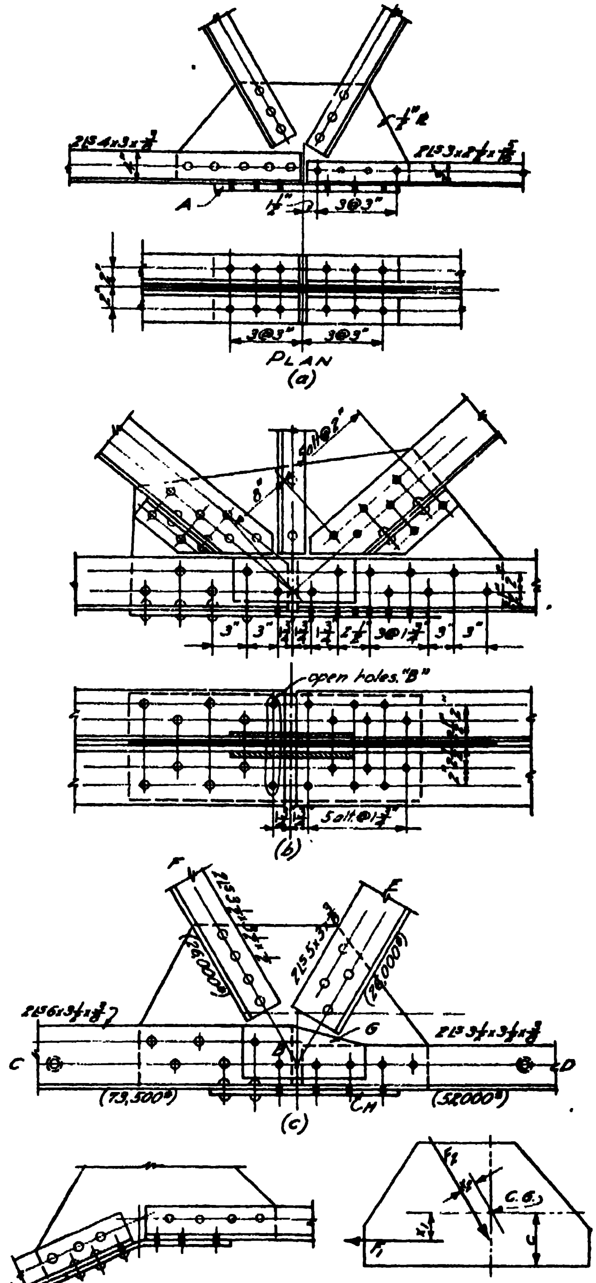


Fig. 336

The proportion of the stress which is to be carried through the splice of the horizontal legs is more or less arbitrarily decided. A good rule of thumb is to divide the total number of rivets required in proportion to the sectional area in each leg of the angle to be spliced.

Some designers provide for any bending stresses induced in a gusset plate of an unusual size. In most cases, the center of gravity of the gusset plate does not coincide with the working point of the joint. The forces on either side of the splice tend to rotate about the centroid of the plate. The larger pair of forces will produce the greater bending stress. In Fig. 336 (*e*), let F_1 and F_2 represent the stresses in the members to the left of a splice. Then the resultant moment is

$$M = F_1 \cdot x_1 - F_2 \cdot x_2.$$

The maximum tension may then be calculated by applying $s = \frac{M \cdot c}{I}$, in which I is the moment of inertia of the cross-section of the gusset plate through its center of gravity. This stress may be added to the direct tension in the plate. The combined stress should of course not exceed the allowable.

Illustrative Prob. 214a. Check the arrangement of rivets shown for the joint in Fig. 336 (*c*). Use $\frac{3}{4}$" rivets and a $\frac{1}{2}$" gusset plate. Use the following maximum allowable stresses:

Shear — shop rivets, 12,000#/□"; field rivets, 10,000#/□".
Bearing — shop rivets, 24,000#/□"; field rivets, 20,000#/□".
Tension on net section, 16,000#/□".
Double shear, $\frac{3}{4}$" shop rivet = 10,600#
Bearing, $\frac{3}{8}$" plate = 6,750#

Members BE and BF, $\frac{26,000}{6750} = 3+$. Use 4 rivets.

Member BD
Double shear, $\frac{3}{4}$" field rivet = 8840#
Bearing, $\frac{3}{8}$" plate = 5630#
$\frac{52,000}{5630} = 9+$. Use 10 rivets.

Since the member is of equal legged Ls, $\frac{1}{2}$ of 10 rivets should be in each set of legs. For practical reasons, use 4 in vertical legs and 6 in horizontal legs, as shown.

Member BC. Two field rivets in each leg should be the minimum to accommodate the plate G and allow adjustment in the field for the plate H so that the member BD can be slid into place.

4 field rivets @ 5630 = 22,500#
73,500 − 22,500 = 51,000#
$\frac{51,000}{6750} = 7+$ shop rivets req'd in addition.

Use 8 shop rivets.
4 field rivets.

The batten plates, G, are used to provide lateral stiffness.

In Fig. 336 (*e*), suppose $x_1 = 5''$ and $x_2 = 2\frac{1}{2}''$, gusset plate 17" deep, and $c = 8''$.

$$M = 73,500 \times 5 - 26,000 \times 2\tfrac{1}{2} = 303,000''\#$$

$$I = \frac{0.375 \times (17)^3}{12} = 159''^4$$

$$s = \frac{303,000 \times 8}{159} = 15,200\#/\square''$$

Tension on net section of gusset

$$\frac{73,500}{17 \times 0.375} = 11,500\#/\square''$$

Combined stress = 26,700#/□" (excessive)
A $\frac{1}{2}$" gusset should be used.

In practice, this latter calculation is not usually made, and a reasonably heavy gusset plate is used arbitrarily.

Prob. 214b. Design and detail at a scale of $\frac{3}{4}$" = 1'-0", the joint in Fig. 337 (*a*). Use working stresses given in Illustrative Prob. 214*a*. Members AB and AC are to be connected.

Prob. 214c. Determine the required number of rivets and the splice material necessary for the joint shown in Fig. 337 (*b*). Member XY is to be field connected. Use the customary allowable stresses.

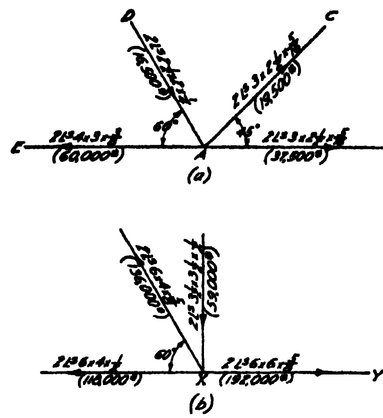


FIG. 337

215. Bearing at Wall Supports.

When a roof truss rests upon a wall, the maximum end reaction must be provided for by a bearing area sufficiently large so that the allowable bearing value of the masonry is not exceeded. This is usually done with a **sole plate,** riveted to the end of the truss, as illustrated in Fig. 338.

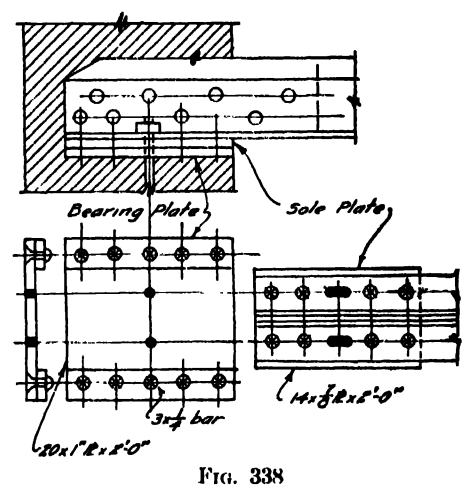


FIG. 338

This helps to distribute the load in the bottom chord angles over the bearing in a more uniform manner. The sole plate, in turn, rests upon a bearing plate, placed in the wall. The widths of these plates should not be less than that of the bottom chord angles of the truss, and they should, preferably, not project more than 3" or 4" either side of the angles. Provision for anchor bolt holes is often necessary. The steps in the design to determine the sizes of

are similar to those required for bearing for steel beams (Art. 15).

Illustrative Prob. 215a. Determine an arrangement at the bearing of a truss on a 16" brick wall if the maximum end reaction is 38,000#. Maximum allowable bearing pressure = 200#/□". Bottom chord angles 6" × 4" × ⅝", ½" gusset plate.

$$\text{Area required} = \frac{38{,}000}{200} = 190\square''$$

$$\text{Available bearing length} = 16 - 4 = 12''$$

$$\text{Width required} = \frac{190}{12} = 15+. \quad \text{Use 16'' width.}$$

$$\text{Actual pressure} = \frac{38{,}000}{12 \times 16} = 197\#/\square''$$

Try ⅝" sole plate.

Calculate bending moment at A in Fig. 339, the toe of the fillet of the angle.

$$M_A = \frac{197 \times (6.75)^2}{2} = 4630''\# \text{ (For a 1'' strip).}$$

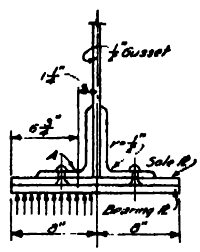


FIG. 339

The sole plate is assumed to be riveted to the angles with a sufficient number of rivets so that their combined thickness may be counted upon as one.

$$M_r \text{ of 1'' thickness} = \frac{16{,}000 \times 1 \times (1)^2}{6} = 2660''\#$$

Moment to be carried by bearing plate = 4630 − 2660 = 1970"#

$$1970 = \frac{16{,}000 \times 1 \times t^2}{6} \qquad t = 0.86''.$$

Use a 12 × ⅞ × 1'-4" bearing plate.

A ¾" sole plate and a ¾" bearing plate might also be used. The thickness of the sole plate is limited to the diameter of the rivets to obtain easy punching (Art. 23). If the bearing plate thickness exceeds 1½", a rolled steel slab or a grillage would have to be used (see Index). Several combinations may be used for ordinary cases, but the metal should be "balanced," that is, the thicknesses should be somewhere near alike. For light reactions, no bearing plate may be theoretically required, but a ½" plate should always be used, as a minimum requirement.

When large trusses are subjected to large temperature variations, there is a considerable change in length. This may be calculated, approximately, by applying the formula

$$e = k \cdot l \cdot T, \text{ in which}$$

e = the total change in length, in inches,
k = the coefficient of expansion of steel per degree Fahrenheit = 0.0000065,
l = the length of the truss in inches, and
T = the total temperature range selected, in degrees Fahrenheit.

Thus for a 10'-0" length, and a change of 150° F., $e = 0.0000065 \times 10 \times 12\ (150) = 0.117''$. Based upon similar calculations, a rule of thumb sometimes used is to allow **⅛" for every 10'-0" of span.**

For spans up to 80'-0", the "sliding plate" detail, or in other words, the sole plate resting upon the bearing plate, as illustrated in Fig. 340, is usually

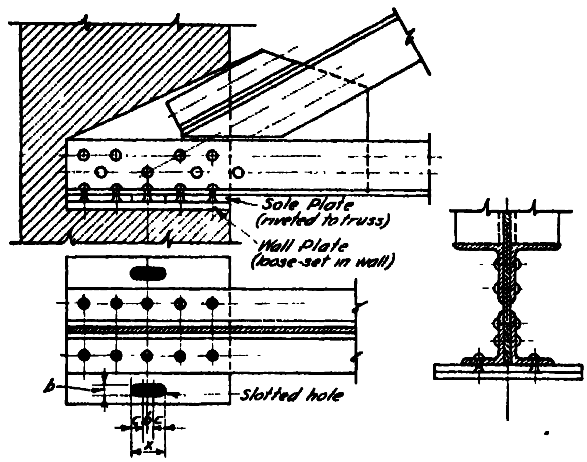


FIG. 340

sufficient. When anchor bolts are to be used, the holes for them may be **slotted,** as shown. The length, c, should be made at least to conform with the above rule. **Holes for anchor bolts are commonly made $\frac{5}{16}$" greater than the diameter of the bolts.** This allows adjustment of the bolts to the truss when it is erected, — the bolts having been previously set in place by the mason. The length, x, in Fig. 340, is then $2c + b$. This should be dimensioned to the nearest ¼". The expansion end of a truss is also an aid, if horizontal thrust is exerted. This is an actual provision corresponding with the theoretical assumption made, — that one end is "on rollers" (Art. 187). When no anchor bolts are used, the horizontal end reaction (when one occurs) should not exceed a value of the vertical reaction times the coefficient of friction at the plane of support. The value of the latter for steel resting upon steel is about 0.15. If the horizontal component is larger than the sliding resistance, the shearing value of the anchor bolts must be considered.

For very large trusses, where the spans exceed 100'-0", which are rather large for building work, flat plates are not very efficient, and some form of **rollers,** or a rocker, may be used.* When both ends

* In some cases, it is possible that wind shear may nearly all come upon one wall, due to unequal bending in anchor bolts or due to temperature movements. For large spans, rollers are an advantage in relieving such action.

of a large truss are fixed, the truss may also be pivoted at the apex. The strength of roller bearings has never been very conclusively determined. The following empirical formulas were established some time ago at Cornell University:

$p = 1200\sqrt{D}$, and

$l = \dfrac{P}{p}$, in which

p = the allowable load in # per lineal inch of roller,
D = the diameter of the roller in inches,
P = the total load to be carried, in #, and
l = the length of the roller in inches.

A typical detail is illustrated in Fig. 341. The formulas are based upon the assumption that the load is uniformly distributed over the lengths of the rollers, and upon a factor of safety of 5, if the plate is of cast material; or 3, if the plate is of steel. The elastic limit of the material is considered as the

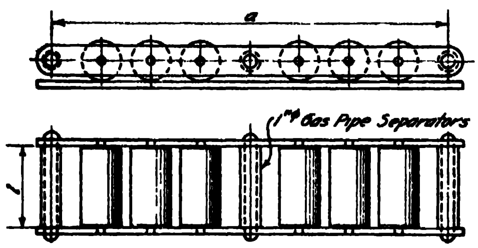


Fig. 341

governing ultimate strength. Some designers prefer to use a higher factor of safety and employ the formula $p = 600\sqrt{D}$, to protect against defects, poor workmanship, and conditions of uneven bearing. The first formula is recommended as being reasonably conservative for use. The length of the rollers is determined by the width of the plate parallel to the wall, and the number of rollers, by the available bearing length. The diameter of the rollers will be fixed by the implied conditions. It is obvious that there is considerable " cut and try " to such a design.

Illustrative Prob. 215b. Determine an arrangement of roller bearings for an end reaction of 86,000#, if the maximum allowable bearing on the masonry is 300#/□". Wall bearing = 16".

Try 3" rollers. $p = 1200\sqrt{2} = 1700$#/inch.

Area required for bearing plate $= \dfrac{86{,}000}{300} = 286$□"

$\dfrac{86{,}000}{1700} = 51''$ $\quad \dfrac{286}{16} = 17.9$ $\quad$ Use 18".

$\dfrac{51}{18} = 2.8$ $\quad$ Use 3-2"ϕ rollers — 18" long.

These may be placed transversely in the 16" bearing length easily.

For very heavy trusses, a bearing of the swinging arm type, or **rocker**, may be used, as illustrated in Fig. 342. A trunnion is used at *A*, which is cupped to prevent too much rotation. Rollers are placed under the bearing plate *B*. An advantage of an end detail of this kind is that deflection of the truss will not cause an unequal pressure on the bearing and a consequent overload on the rollers, if used under the bearing.

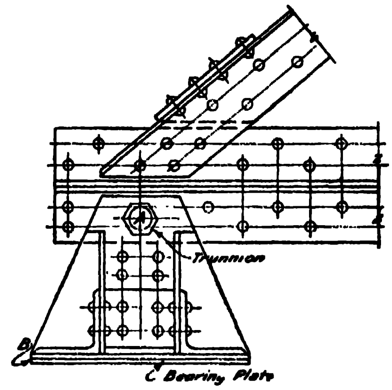


Fig. 342

Anchor bolts are sometimes used at the ends of trusses. They are theoretically required when there is an uplift exerted at the support, due to wind loading, or other causes. In this case, the required diameter at the root of the threads may be computed, based upon an allowable tensile stress of 16,000#/□". Anchor bolts are often employed when no uplift is probable, their size being arbitrarily made 1"ϕ. The holes are usually made $\frac{5}{16}$" greater than the diameter of the bolts, to allow adjustment as illustrated in Fig. 340. They should extend a sufficient distance below the seating plane of the truss to engage a reasonable amount of masonry. A value of 2'-0" is commonly used. Plate washers should be used at their lower ends to develop the tension in the bolts, as illustrated in Fig. 344.

Prob. 215c. Determine an arrangement at the bearing of a truss on a brick wall if the maximum end reaction = 34,000#. Maximum allowable bearing pressure = 250#/□" and wall bearing = 8". Bottom chord angles 5 × 3½ × ½ and ⅝" gusset plate.

Prob. 215d. Arrange a bearing for the heel of the truss shown in Fig. 335 (*a*) if the wall bearing is 10" on a concrete wall (p = 500#/□"). ½" gusset plate.

Prob. 215e. What size of slotted holes should be used in Prob. 215c if the anchor bolts are to be 1"ϕ and the span of the truss is 80'-0"? Use a temperature range of 120°. How does the rule of thumb compare with the theoretical values?

Prob. 215f. If a horizontal thrust of 11,000# is exerted at the end of a truss bearing on a sliding plate, with no anchor bolts used, and the maximum end reaction is 51,000#, is the detail satisfactory? If not, what size of anchor bolts would be theoretically required?

Prob. 215g. Determine an arrangement for roller bearings for an end reaction of 112,000# if the maximum allowable

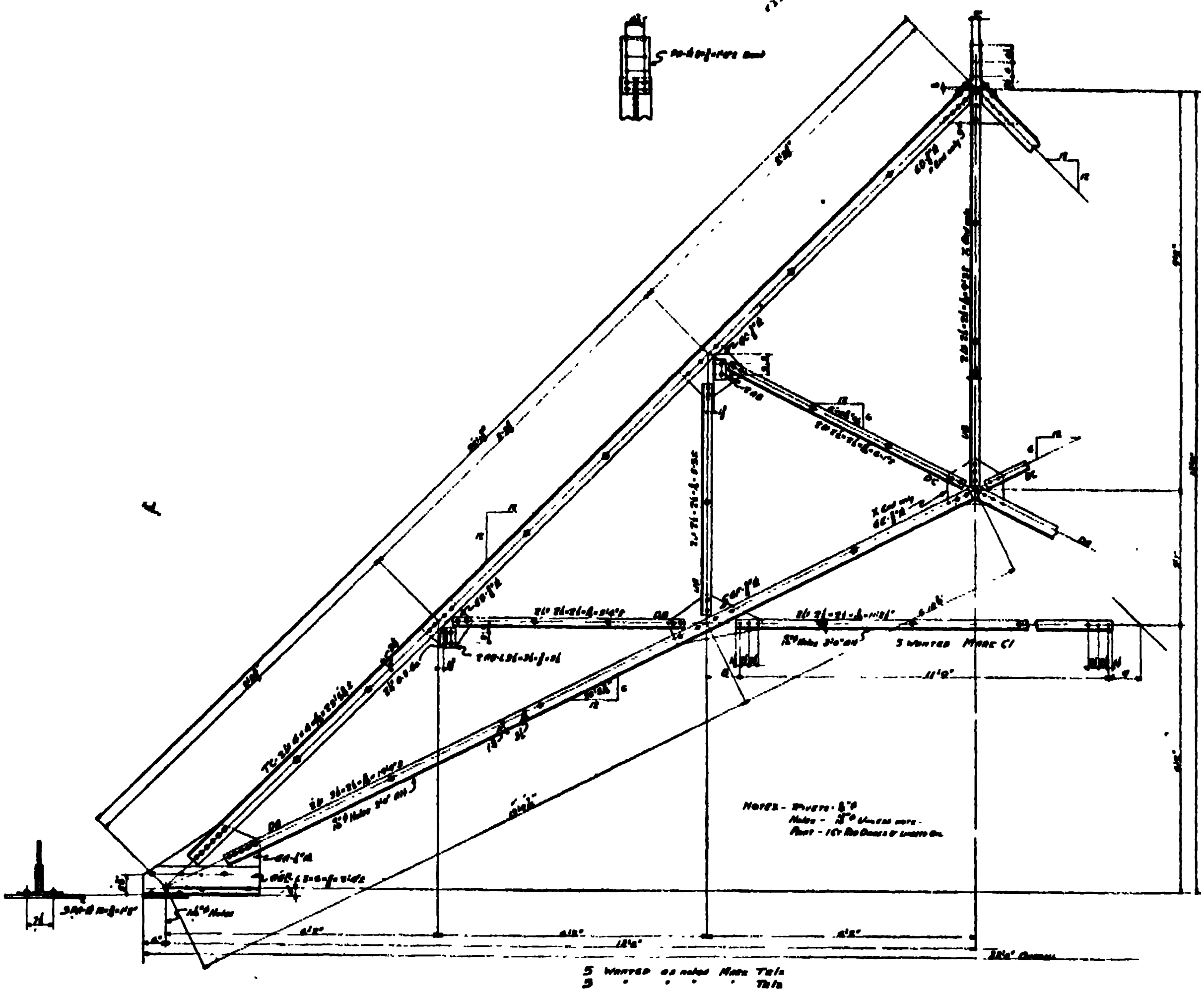


FIG. 343. STRUCTURAL STEEL DETAILS*

bearing pressure on the masonry is 250#/□″. Wall bearing length = 16″.

Prob. 215h. What size of anchor bolts (use two) is theoretically required if a truss is subjected to an uplift of 21,000#? What practical size should be used? What sized holes should be provided for them? What size of plate washer is required if the allowable pressure on the masonry is 200#/□″?

216. Purlin Connections.

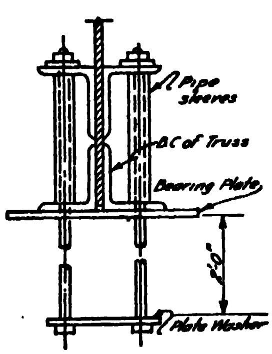


FIG. 344

Rolled structural steel channel sections are most commonly used for the purlins in steel-framed roofs. They are more readily attached to the trusses, and also allow the convenient use of nailing strips. While channels are less resistant to lateral bending than some of the other structural shapes, usually the details of the roof construction are made to offset this tendency by introducing tie-rods. The design of such members has already been discussed (Art. 170). The purlins usually receive only one-half a panel load at the ridge and at the eaves. In order to maintain a uniform roof surface, the depth is kept the same as the intermediate purlins, although they may be of a lighter weight. If tie-rods are used, the purlins at the ridge must develop the normal components of the stresses in the tie-rods. Sometimes the purlins are used as struts in a bracing system (Art. 198), in which case they should be liberally designed. The channels are more effective if they are faced with the flanges up the slope of a roof. In this way, the clip connection angles are down the slope, acting as a shelf support. Occasionally, the channels are reversed if spiking pieces are to be used.

* Courtesy of the Eastern Bridge and Structural Co.

Figure 345 illustrates several types of purlin connections. These are usually bolted in the erection, so that the connection angles are usually "bolted complete" to the truss when it is shipped, — that is, the angles are bolted to the trusses, and the purlins are shipping pieces with open holes only. One bolt hole, K, is ordinarily provided in the upstanding leg of the clip angle for each connection. Two should be provided if the purlin is to serve as a strut also. Large purlins (10″ and 12″ channels) are usually connected with bolts through the flanges also, such as those at C in (a). The leg of the clip angle bearing against the web of the purlin should be long enough to reach well up and give the purlin as much web support as possible (usually 5″ or 6″ legs are used). Two-bolt connections are commonly made as shown by B in Fig. 345 (a). The purlins receive some shear from the component of their load, tending to slide them down the roof. The bolts connecting the angle to the truss must have sufficient sectional area to resist this shear. Spiking pieces may be bolted to the channels, as illustrated, when plank roofs are used.

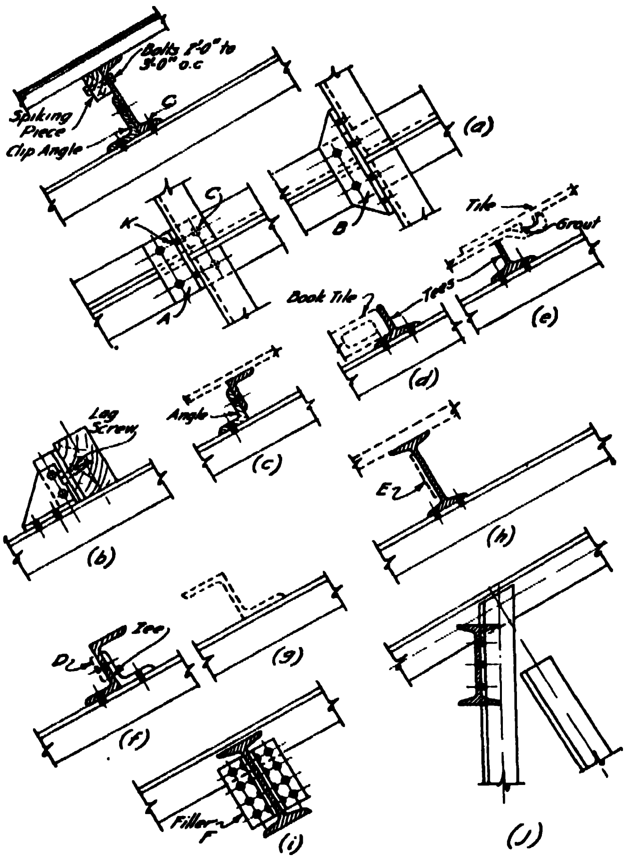


FIG. 345

Figure 345 (b) shows a detail which may be employed when wood purlins are used. Angle purlins are often supported as shown in (c). The angle possesses greater stiffness when the outstanding leg is turned up the slope. T-purlins usually may be connected directly, as illustrated in (d) and (e). Z-bars are commonly fastened in a similar manner, as shown in (f). A splice plate may be used, such as D, or a clip angle, may be employed, as is shown dotted. The Z-bar should not be turned as indicated in (g).

I beams are sometimes used as purlins when the spans are long, or when a greater resistance to lateral bending than a channel will supply is required. They are usually connected directly to the top chord, as illustrated in Fig. 345 (h). A splice plate, E, may be used, if necessary. If it is desired to keep the purlin in the plane of the top chord, standard connection angles (Art. 28) are used by supplying fillers at F in (i), placed on top of the gusset plate. In monitor framing, and so on, the purlins are often framed into a vertical web member, as illustrated in (j).

Prob. 216a. If the inclination of the top chord in Fig. 345 (a) is 30° with the horizontal, and the vertical end reaction of the purlin is 8000#, is a one-bolt field connection sufficient? Use 8000#/□″ shear for bolts and 16,000#/□″ bearing.

217. Bracing Details.

The general discussion of the purposes and types of truss bracing has already been given (Art. 198). This is usually effected in structural steel work by angles or by rods. The former are more commonly used in ordinary roofs, as they are more rigid and require simpler connections. Rods, however, offer less surface to corrosion and are used considerably in mill buildings (Chap. 30). In the latter type of structure, wind stresses in a roof may be approximately computed, but in other kinds of roofs, the provision of wind bracing is largely a matter of good engineering judgment, and less is required in the usual roof because the roof carrying materials supply considerable stiffness in themselves. Bracing is of course also useful in resisting vibration and stresses due to erection. Experience has shown that certain minimum sized angles are usually sufficient and provide reasonable connections. Top chord bracing is often made of $3 \times 3 \times \frac{5}{16}$ angles. Bottom chord bracing may be lighter, and $3 \times 2 \times \frac{5}{16}$ angles are common. Ordinary bracing points are at the center line and at the quarter points of the span.

Fabricating companies usually plan to introduce some initial tension in the bracing members by making the distance center to center of end holes $\frac{1}{8}$″ less than the theoretical distance, $\frac{1}{16}$″ of this to take up any play and the other $\frac{1}{16}$″ to introduce

the initial tension. Figure 346 illustrates typical bracing details. The number of field rivets used is commonly made sufficient to develop the net section of the angle at the working stress, say $16{,}000\#/\square''$. The necessary gusset plates are planned to accommodate the rivets in the usual manner. For light work, $\frac{1}{4}''$ plates are employed, and in heavier framing, $\frac{5}{16}''$ plates are used. The latter provide a better rivet resistance in bearing.

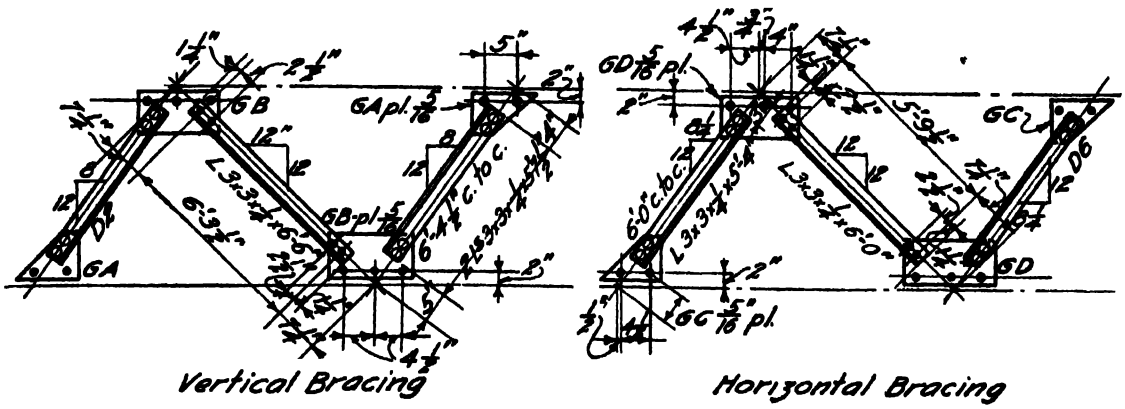


FIG. 346

A member is commonly used at the vertical center-line of a truss for practical reasons, even if it is not theoretically required. This is often called a **sag tie,** as it prevents the bottom chord from excessive deflection. From an erection standpoint, this member is valuable, as a truss may be assembled on the ground and raised into position by a locomotive crane or gin pole, using the hanger as a hitch. This throws the bottom chord of the truss temporarily into compression. Workmen may also attach a block and tackle to the truss after it is in place for purposes of erecting other work.

When the purlins at the peak of a truss do not offer sufficient bracing resistance, or a monitor or other similar construction occurs, a **ridge strut** may be used. This consists of a pair of angles framing between the trusses at the peaks in the braced bays (Fig. 330 (*b*)), and a single angle in the intermediate unbraced bays.

CHAPTER 18

MISCELLANEOUS TRUSSES

218. Cantilever Trusses.

In grandstands, sheds, railway stations, and so on, portions of the trusses are sometimes projected beyond the supports. Such trusses are commonly called cantilever trusses. Figure 347 shows diagrams of some of the types. The projecting arms should usually be confined $\frac{1}{4}$ to $\frac{1}{3}$ of the anchor spans.

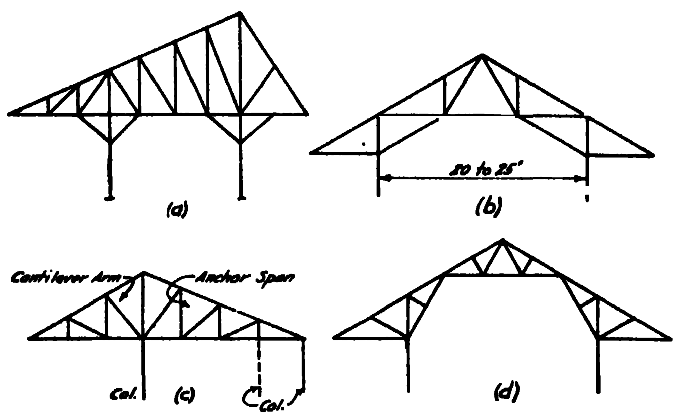


FIG. 347

The common procedure in determining the stresses may be employed (Chap. 15) although a careful check-up of the analysis should be made. The stresses in the projecting portions are reversed from those in typical trusses with a simple span,— tension occurring in the top chord, and compression in the bottom chord, which is characteristic of the cantilever. Typical diagrams are given in Fig. 348, which show the general application. Care should be used in analyzing the uplift on interior columns.

Prob. 218a. Lay out the truss shown in Fig. 348 (*a*) to a larger scale and obtain the values and kinds of stresses in all the members. Height of truss = 10'-0".

Prob. 218b. Repeat Prob. 218*a* for Fig. 348 (*c*). Inclination of top chord = 30°.

219. Trusses for Supporting Floor Loads.

Occasionally, trusses are used to carry a superstructure over a large opening underneath it. An instance of this kind occurs when a large floor space is desired in the first story, such as for lobbies and the like, and the upper stories are subdivided for offices or apartments. In this case, the truss is made as deep as the second story height so that the second and third floor frames coincide with the bottom and top chords respectively. Door openings and the like may be planned to occur between the diagonals of the web system. A truss of the Pratt type, as shown in Fig. 349 is usually employed for such work, with parallel top and bottom chords. Figure 350 shows an application of a truss for supporting several floors.

220. Sawtooth Trusses.

In certain types of mill construction, a sawtooth roof, similar to that illustrated in Fig. 351, offers features which are considered to be advantageous. The important gain is an abundance of uniformly diffused light. Other advantages are the available headroom and the economy in lighting charges. Some disadvantages are the possibilities of excessive condensation under the roof, poor ventilation, leaks, and excessive heat. Proper details and careful workmanship must be insisted upon to eliminate the above disadvantages. The cost of such construction exceeds that for flat roofs, as the amount of ordinary roofing is practically the same, and the glazing, gutters, and such details, are additional in the sawtooth roof. The advantages gained very often warrant its use, however.

The usual plan is to avoid the direct sunlight by placing the steep pitch planes toward the north. The glass is set an angle of about 20° with the vertical in order to obtain the brighter light of the upper sky and to prevent cutting off the light from the other sawteeth. Such an angle also assures a uniform diffusion of light over the floor below. Double glazing is preferred but not necessary. This provides a space between the layers of glass to offset the conductance of heat and hence eliminates some of the condensation. The glass should be of the factory ribbed type with the ribs placed vertical and facing in. The first provision eliminates the glare of the light and the second minimizes the effects of the shadows cast by the adjacent sawtooth. A wire netting is commonly placed under

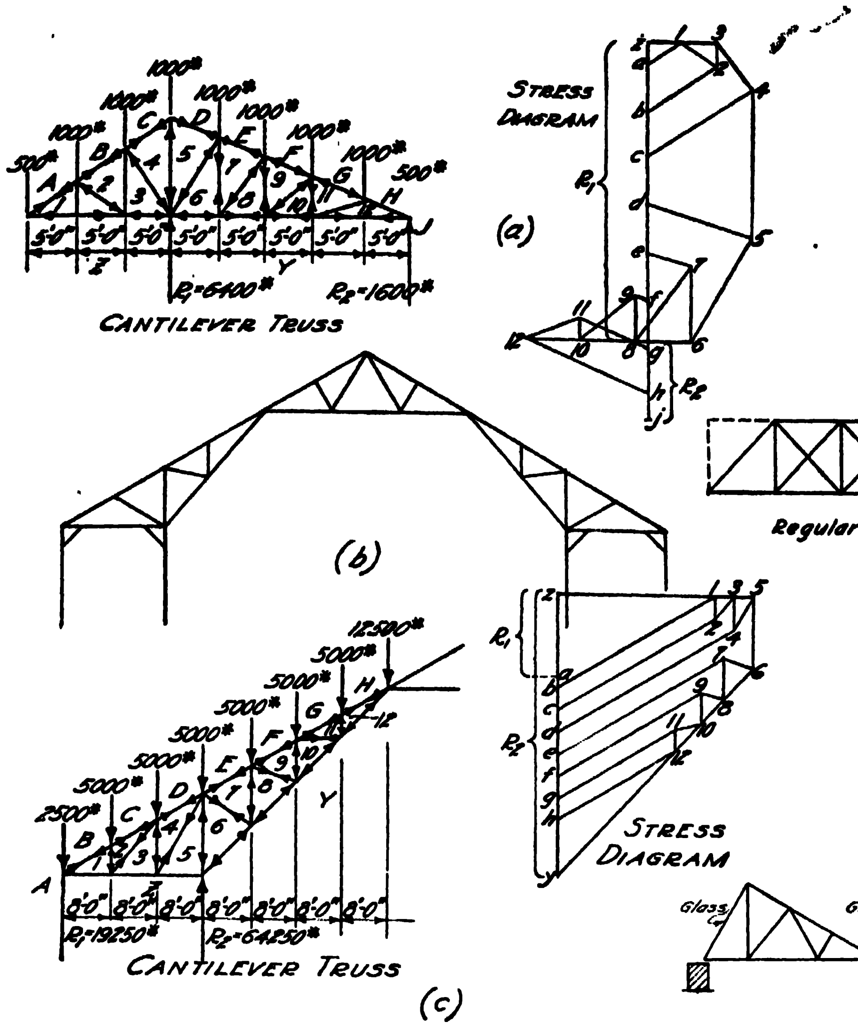


Fig. 348

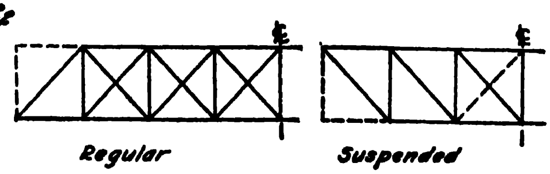


Fig. 349

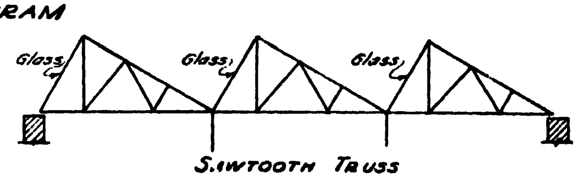


Fig. 351

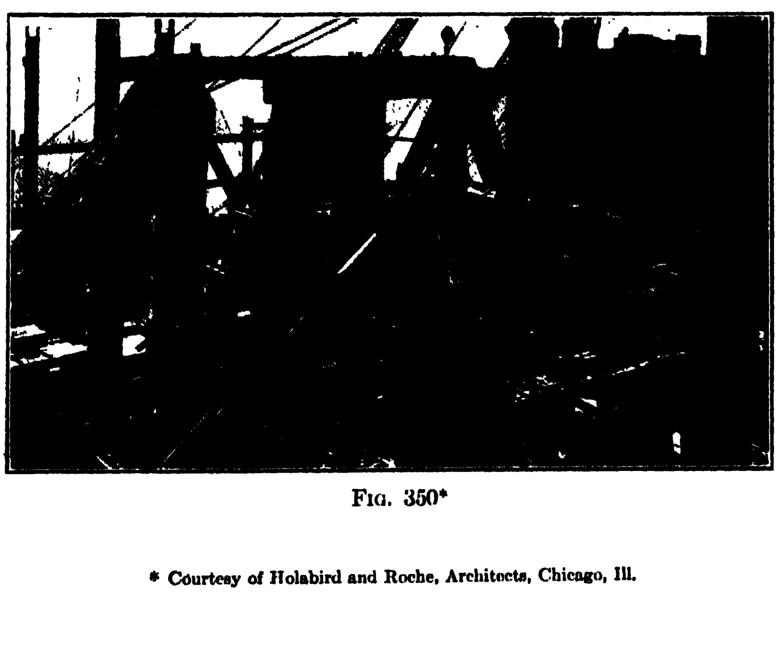


Fig. 350*

* Courtesy of Holabird and Roche, Architects, Chicago, Ill.

the sash to prevent any broken glass from falling on to the floor below. Condensation gutters with inside conductors are placed under the truss in the interior. The conductors should not be led to the outside under the sash, as this leaves openings to admit cold air and is likely to cause trouble in freezing weather. The portion of the roof between the sawteeth is drained by providing valleys 14″ to 20″ wide, pitched about ½″ per foot to conductors placed about 50′-0″ apart.

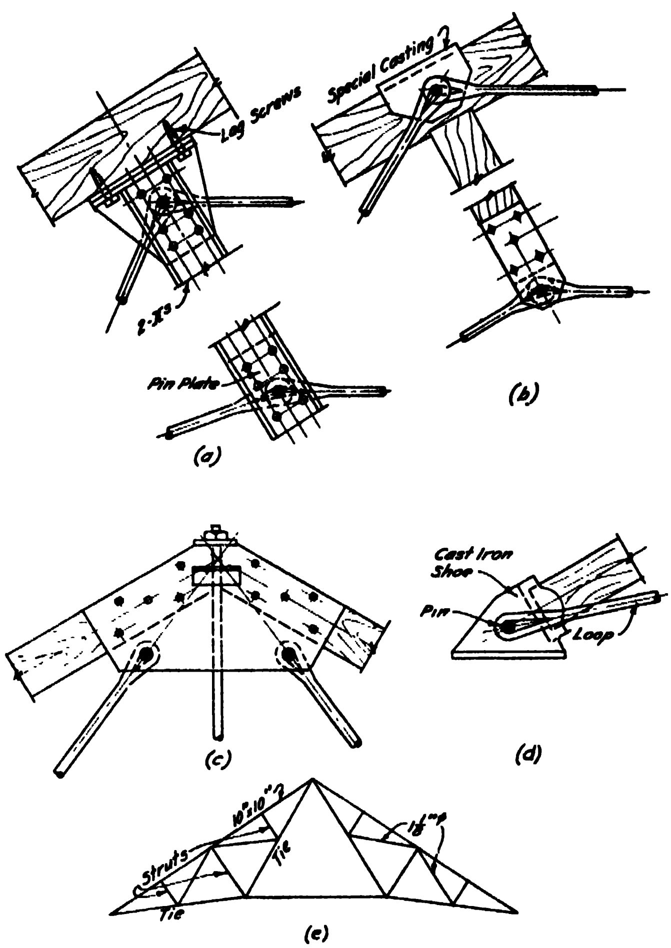


Fig. 352

For small roofs, 3″ plank is used as the roof supporting material. Interior sheathing, if used at all, should be placed directly under the plank, with no concealed air spaces. Figure 351 illustrates the common web system used. The design of the trusses is similar to that previously discussed.

221. Composite Trusses.

Roof trusses made with the top chord of timber, the tension members of steel bars, and the web compression pieces of timber or of structural shapes, were employed to some extent in former practice, but in modern construction, their use is practically discontinued, because trusses completely of structural steel are more economical and more desirable. However, occasional instances might make the use of such a composite truss advantageous. They are usually of the scissors, Howe, or Fink types (Art. 182). Some advantages claimed for such trusses over wood trusses are that less shadows are cast, there is less chance for accumulation of dirt, less obstruction of light, and that the joints are simpler. Figure 352 shows some typical details.

The strut connections are similar to those used for wood trusses (Book I). The tension members may be pin-connected (by special castings or directly to the timber). They may be made with nuts either in square bearing, or by the use of special castings or beveled washers, as illustrated in Fig. 353 (*a*). When a truss is to be pin-connected, nuts cannot be used and a clevise usually is employed, as shown in (*c*) and (*e*). For small diameter pins, the forks may be straight. For pins of large diameter, the forks should be closed in so as not to overstrain the pin. Sometimes rods are looped by bending and welding the ends to the main length, as shown in Fig. 353 (*c*). It is almost impossible to connect light steel tension members to timber securely without allowing some small movement. Consequently, **turnbuckles,** or **sleeve nuts,** as shown in (*d*), may be used intermediately in the members to allow adjustment. Right and left threads must be used as shown, so that the nut or turnbuckle will tighten both bars, or a swivel may be employed.

222. Arched Trusses.

An arched truss is one which has the general shape of an arch, but which may be analyzed by the usual procedure common for trusses. Rollers are used at one end, as for any large truss, to allow horizontal motion. For large open roofs, such as in train sheds or auditoriums, arched trusses are often used, as they are economical and offer a pleasing appearance. A greater clear center height is possible and erection costs are less, as less scaffolding is required. Such trusses are adaptable to curved roofs, as no economy would be served for straight pitched roofs, because uprights would have to be built up to support the purlins in the latter.

The principal compression members follow the greatest lines of stress and hence the bracing members are light. If the joints of the top chord were in the line of a true parabola with the lower ends connected by a tie, and uniformly loaded, there would be no stress in the web system, theoretically. However, loads are seldom uniform, and web members are of course necessary. Arched trusses are satisfactory up to 100′-0″ spans, but for spans in excess of this figure, some type of trussed arch, usually three-hinged (Art. 224), is more economical and desirable, as contraction and expansion are more readily provided for.

Figure 354 (*a*) shows a crescent or bowstring truss. In this type, for spans less than 75′-0″, the uprights are usually radial. For greater spans, they

are commonly made vertical. These trusses are generally built on the arcs of circles, the radius varying from $\frac{1}{2}$ to $\frac{3}{4}$ of the span, and the depth of the truss at the center line is often made equal to one-half of the radius. It is theoretically not a simple truss nor a true arch, but the stresses are sufficiently correct when obtained by the usual truss analysis. Figure 354 (*b*) is a real arched truss and in reality is a series of segmental arched ribs. The type in (*c*) is called a quadrangular truss. It is really two trussed rafters held together by a tie-beam. Figure 354 (*d*) and (*e*) shows other variations in arched trusses, that in (*e*) providing supports for intermediate purlins. The members in all arched trusses are usually made straight between joints. This reduces the cost of fabrication, and avoids complicated stress analyses, for when curved members are used, their axes are not coincident with the action of the forces and the members tend to deform, thereby inducing secondary stresses.

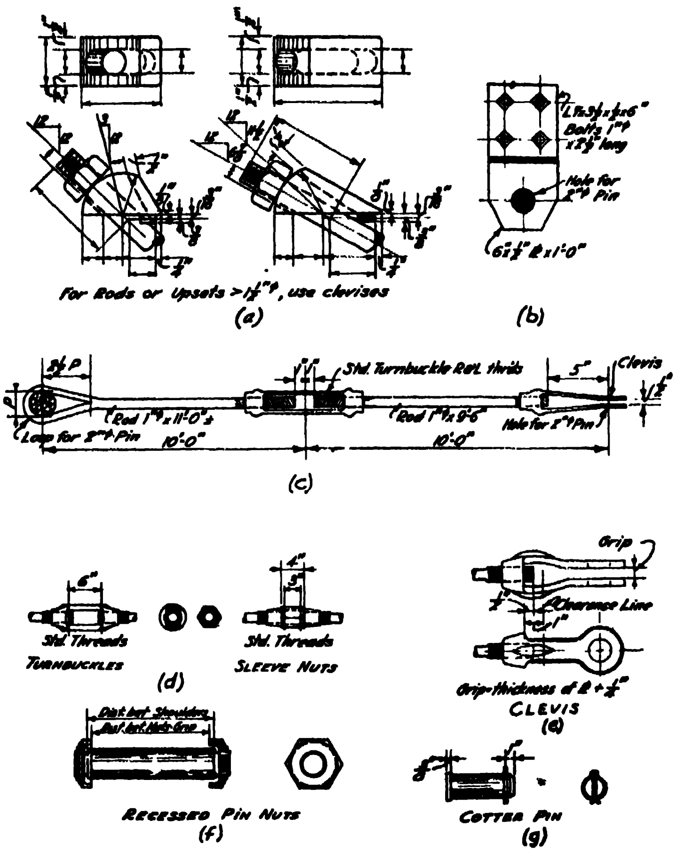


Fig. 353

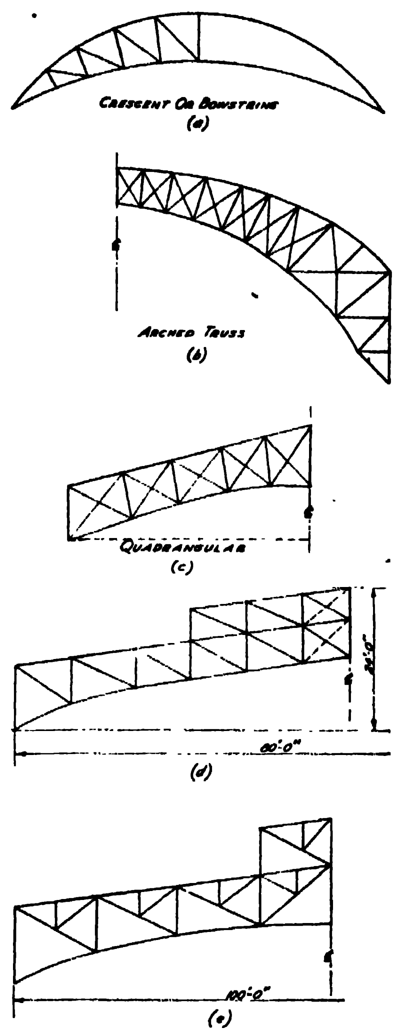


Fig. 354

223. Trussed Arches.

There are three types of trussed arches, namely,

(1) the three-hinged,
(2) the two-hinged, and
(3) the fixed.

The first of these is the simplest to analyze and is the most commonly employed. The two-hinged arch has some special advantages and is occasionally used. The third type has no hinges and is seldom employed in the form of a truss for structures. The nearest approach would be a plate girder bent to the form of an arch. It is very complex to analyze and is similar to that of an arch with solid ribs built of masonry.

224. Three-Hinged Arches.

For buildings such as armories, exhibition halls, auditoriums, train sheds and the like, where large open spaces are desirable, three-hinged arches are economical. No interior columns are required. Good resistance to wind pressure is also obtained. The three-hinged arch is advisable where temperature variations are concerned, as the center is free to rise and fall. Figure 355 shows an outline of a

typical arch of this kind, the hinges occurring at x, y and z. In some cases, the arch is surmounted by a lantern. The arch is composed of two separate parts, each a form of semi-circular arch, and acting as an individual unit. Each depends upon the opposing force from the other to maintain the equilibrium of the system.

In general, the usual principles of truss analysis are sufficient to determine the stresses, — at least accurately enough for practice. As in any truss solution, the first step is to obtain the reactions. Since the structure is an arch, the reactions are

In a graphical solution for the reactions, an equilibrium polygon permits only the determination of three unknowns. The forces must act through the hinges, however. Hence the theory of passing an equilibrium polygon through three given points may be used. A reference to Fig. 357 will show this method. Lay off the load line, bd, select any pole, P, and draw the rays Pb, Pc, and Pd. The next step is to find a trial closing line. Starting from point x in the space diagram, draw xk parallel to Pb in the space B, then kl in the space C parallel to Pc, and ls in the space D parallel to Pd. Then xs is the trial closing line. Draw PF parallel to xs, in the force polygon. It is known that the final closing line is xz if the equilibrium polygon is to pass through the three hinges. A truth in graphic statics is that pole distances are

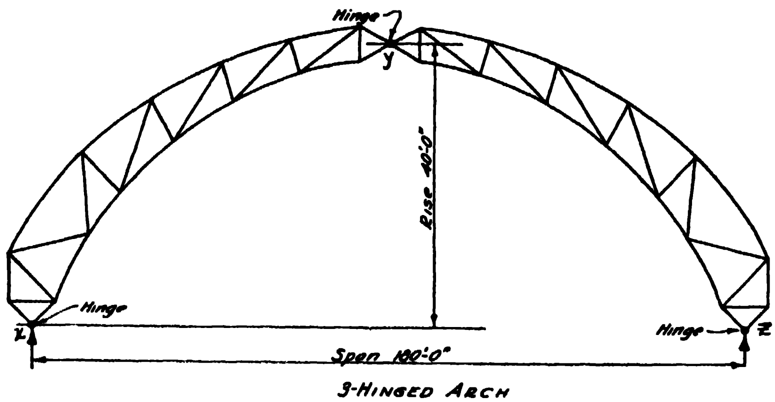


FIG. 355

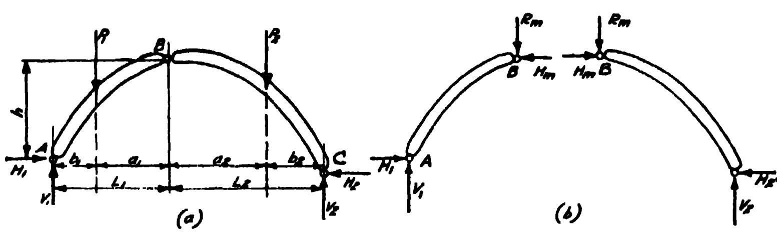


FIG. 356

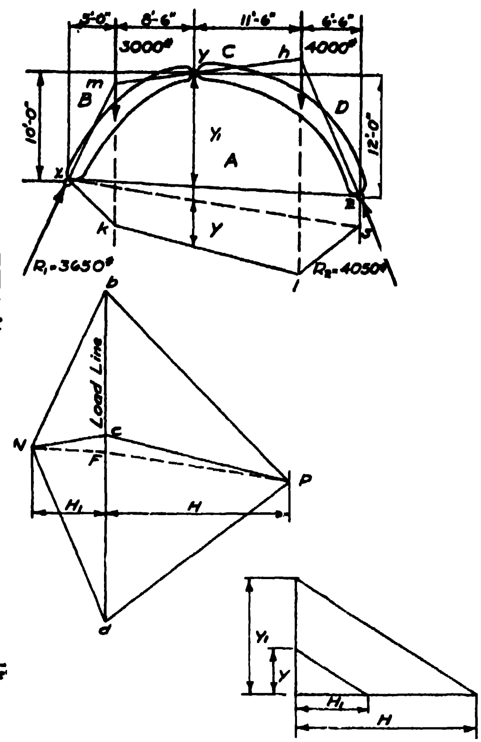


FIG. 357

inclined. This means that there are four unknowns, namely, the amounts and directions of the horizontal and vertical components of R_1 and R_2, respectively. Referring to Fig. 356, the values of V_1 and V_2 may be obtained by considering the structure as a whole and taking moments about A and C in turn. By considering each half of the arch separately and taking moments about B in each case, H_1 and H_2 may be determined. When these four values are known, R_1 and R_2 may be established by combining the respective components. If L_1 and L_2 are equal, as in the usual case, and the arch is symmetrically loaded, the calculations are of course simplified, as $H_1 = H_2$, and $V_1 = V_2 =$ one-half the load. The horizontal thrust around the arch is constant.

inversely proportional to the corresponding intercepts in the funicular polygons, for parallel forces. Then

$$\frac{H}{H_1} = \frac{y_1}{y}, \quad \text{or} \quad H_1 = \frac{H \cdot y}{y_1}.$$

The value of y and y_1 may be scaled from the space diagram, and H may be scaled from the force diagram. When H_1 is known, lay its value off as indicated. Draw FN parallel to xz, and join N with b and d. Then $R_1 = Nb$ and $R_2 = Nd$, to scale. The vertical and horizontal components of R_1 and R_2 may then be determined.

When the reactions are fully established, the stresses in the members of the arch may be obtained in the usual manner. A graphical truss diagram is commonly employed. Figure 358 shows a typical solution. The computations for the reactions are as follows:

By $\Sigma V = 0$,

$$V_1 = 5000 - 5\,(10{,}000) - 5000 = 0$$
$$V_1 = 60{,}000\#.$$

By $\Sigma M = 0$,

$$M_x = -5000 \times 5 + 10{,}000\,(5 + 15 + 25 + 35 + 45) + 5000 \times 48 - 46\,H = 0.$$
$$H = 31{,}850\#.$$

By $\Sigma H = 0$,

$$H_1 = H.$$

(3) wind load on portion to right of center hinge only,
(4) snow load on portion to left of center hinge only, and
(5) snow load on portion to right of center hinge only.

It is reasonably conservative not to consider the wind and snow loads as acting on the same portion of the roof simultaneously, because of the curved

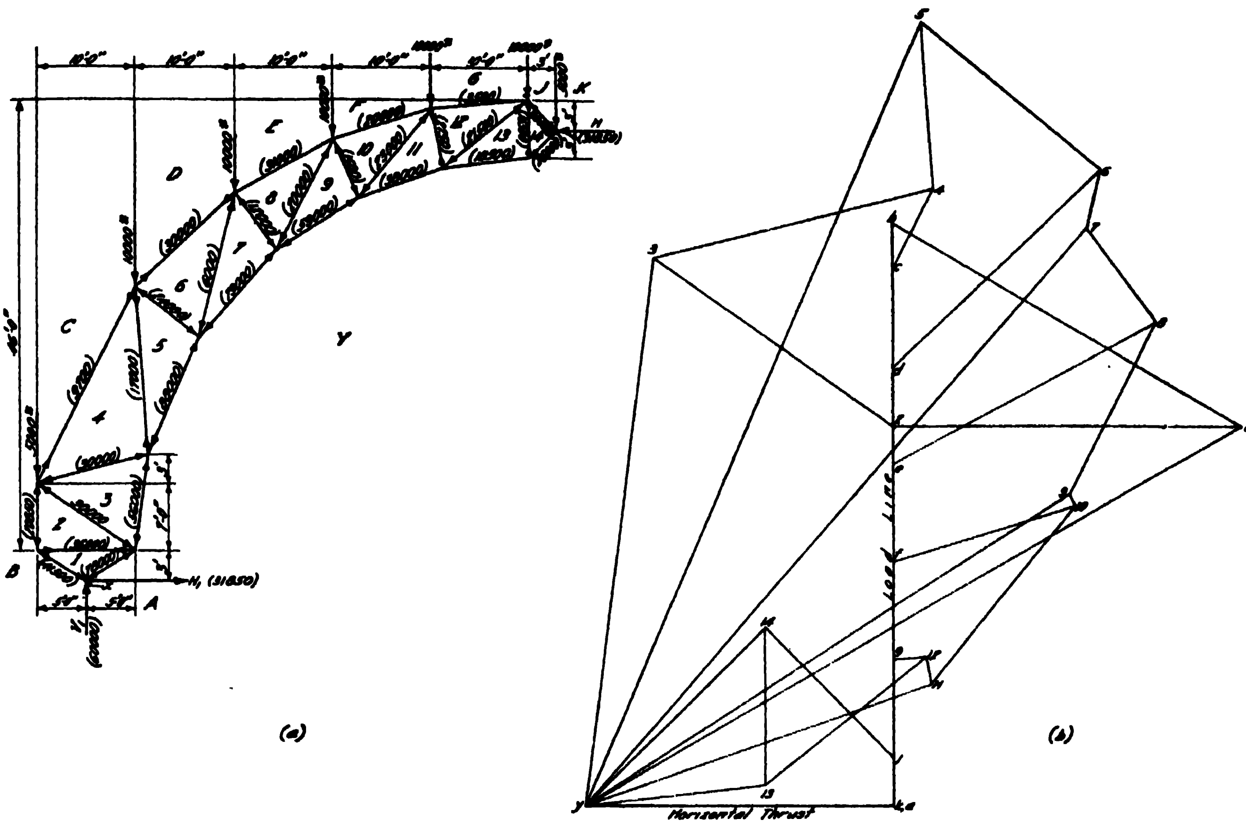


Fig. 358

roof, upon which snow would not easily remain. The snow-load reactions and stresses are obtained in a manner similar to that previously discussed. The wind-load reactions are usually determined by the graphical method (Art. 189). This method is simpler, as the loads are not parallel. When the wind-load reactions are established, the stress diagram may be drawn as in the usual case. Reversals of stress sometimes occur, and this feature should be carefully noted.

The thrust is usually small compared with the values of the other stresses and is either resisted by buttresses built into the walls, or more commonly, by a connecting tie-rod. If the latter is used, it is commonly concealed by placing it just below the floor level.

Figure 358 shows the analysis for vertical loading only. The arch must be designed for dead, snow, and wind loads, as in other trusses. Wind loads are usually assumed to act normal to the roof. In order to determine maximum stresses, combinations of conditions must be tested. The following are commonly assumed:

(1) dead load over whole truss,
(2) wind load on portion to left of center hinge only,

In practice, a horizontal tie-rod is employed to develop the thrust, and rollers are used at one end. This makes the truss action that of a simple frame. If the rollers were omitted, the stress in the tie-rod is indeterminate and would usually result in a great deal of excess section in the rod. The stresses even

in this case would be assumed the same as if rollers had been used.

225. Two-Hinged Arches.

A two-hinged arch is similar in general nature to a three-hinged arch, except that the hinge at the center is omitted. The solution of such a frame is classed as statically indeterminate, as the horizontal components of the reactions depend upon the relative sizes and stiffness of the truss members. The vertical components may be obtained as for any member resting on two supports. The horizontal thrusts may be obtained by employing the formula,

$$H = \frac{\Sigma \dfrac{S \cdot u \cdot l}{A \cdot E}}{\Sigma \dfrac{u^2 \cdot l}{A \cdot E}}. \quad \text{(See Art. 103, Book I.)}$$

The sectional area of each member, however, is unknown at the outset of the design, so that approximations for the areas must be assumed. This results in " cut and try " methods.*

When the horizontal and vertical components of the reactions are established, the stresses in the truss members may be determined by the usual stress diagrams. Temperature affects the horizontal thrust, and the stresses resulting from this source must also be included in the analysis.

226. Pin-Connected Trusses.

Very large and heavy trusses are sometimes made with pin-connected joints (Art. 227). In general, such trusses are confined to heavy bridge work, and they are seldom used for ordinary roof trusses,† especially where the spans are less than 100′-0″. In large hotel work they are now sometimes employed to support upper stories over large ball rooms and banquet halls. In some cases, the main joints are pin connected with some of the smaller ones riveted. Pin-connected trusses do not possess great stiffness unless they are built of very heavy sections. For this reason, a truss of this type usually has to be of a size which requires built-up members. These are generally of plates and angles for the chord members, latticed channels for the struts, heavy eye bars for the web tension members, and occasionally a series of eye bars is used for the bottom chord.

For the common trusses employed in building construction, riveted joints are almost always used, as they are more economical. A riveted truss may be shipped to the site in sections, is erected more easily and more rapidly, and is more rigid, especially for light loads.

The design of the members in pin-connected trusses is carried out in a manner similar to that for other trusses and the main differences between simple trusses and those which are pin-connected are the relative sizes of the members and the design of the joints. The effective length of the compression members and the column formula used should be based on pin end bearing.

227. Pin-Connected Joints.

SPECIFICATION CLAUSES‡

Compression members in pin-connected trusses shall be so designed that the stresses shall not not exceed 75 per cent of the permissible working stress for columns. The heads of all eye-bars shall be made by upsetting or forging. No weld shall be allowed in the body of the bar. Steel eye-bars shall be annealed. Bars shall be straight before boring.

All pin-holes shall be bored true and at right angles to the axis of the members, and must fit the pin within $\frac{1}{32}$ inch. Eye and screw ends shall be so proportioned that upon test to destruction fracture will take place in the body of the member. All pins shall be accurately turned.

Only a simple case of pin-connected joints will be discussed here to cover the exceptional instance which might occur in the design of structures.†

One of the important features in the design of a joint of this kind is to arrange the packing of the strut channels and eye bars so as to obtain the least bending moment on the pin. It is wise to keep the arrangement symmetrical about the center line of the pin, to try to maintain a " balance," and to keep the forces as near each other as possible. The inclined bars at a joint tend to raise the pin, while the struts tend to lower it, so that the object is to arrange the members to oppose each other as much as possible. The size of the pin is not always as important as its rigidity. If the strut channels are placed on the outside ends of the pin, greater stiffness results, but in some cases, greater bending occurs. Usually some intermediate arrangement is used. Figure 359 illustrates this point. Steel fillers are used to " pack " the spaces between the members. Sometimes the center of the pin is placed a little below the working point of the joint to produce a moment which will counteract that caused by the weights of the members.

The size of the pin is determined by principles similar to those involved in the strength of rivets, inasmuch as the pin must be safe in shear and bearing (Art. 27). In addition, the pin must be ample to resist the maximum bending induced in it. The forces on a pin are assumed to act at the center lines of the bearings of the members, and the members are all assumed to be parallel to the central plane of the truss.

Figure 359 illustrates a simple case of a pin-

* The theoretical design of two-hinged arches is quite complex and the general features only are considered here. Methods of procedure are given in Kidder's Architects' and Builders' Pocketbook, — John Wiley & Sons, Inc.

† It is not within the scope of this book to give any comprehensive discussion of pin-connected trusses. Reference may be had to a number of standard treatises of this subject in bridge design texts.

‡ From the Building Code of the National Board of Fire Underwriters, New York City.

connected joint. The maximum horizontal bending moment is that considered as a cantilever, and is

$$30{,}000 \times 6 - 20{,}000 \times 5 - 10{,}000 \times 4 = 40{,}000''\#.$$

The maximum vertical bending moment (caused by the channels in this case) is due to the load con-

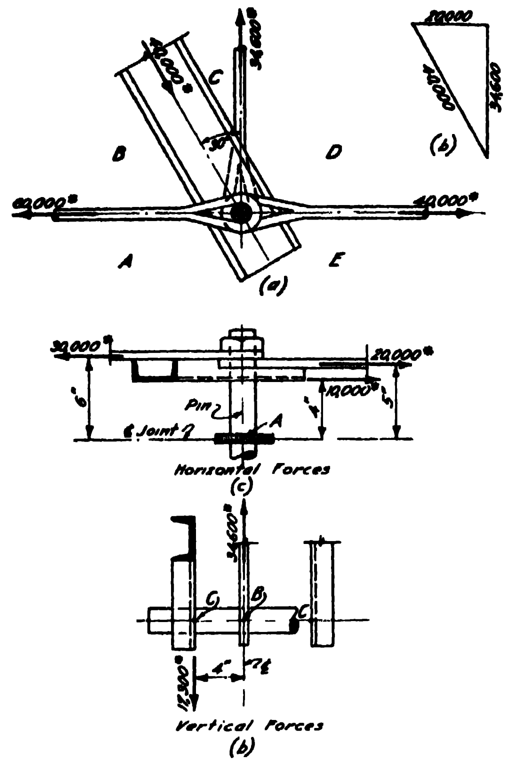


Fig. 359

centrated at the center line of the "span" of the pin, or

$$M = \frac{P \cdot l}{4} = \frac{34{,}600 \times 8}{4} = 69{,}200''\#.$$

The resultant bending moment* is

$$\sqrt{(40{,}000)^2 + (69{,}200)^2} = 80{,}000''\#.$$

The moment of resistance of the pin is that of a solid cylindrical beam. From this, the size required for bending may be obtained, or

$$M = \frac{s \cdot I}{c} = \frac{s \cdot \pi \cdot d^3}{32}, \text{ from which}$$

$$80{,}000 = \frac{24{,}000\dagger\ \pi \cdot d^3}{32}, \text{ or } d = 3\tfrac{15}{16}''.$$

Pins are usually made of cold-rolled shafting, and the odd sixteenth inch sizes are standard, such as $2\frac{15}{16}''$, $3\frac{1}{16}''$, etc.

* Graphical methods may also be used, but these are not as common, and are generally more complicated to analyse.

The maximum shear is developed by the greatest lateral force. At any point this may be expressed by $\sqrt{H^2 + V^2}$, where H and V are the greatest horizontal and vertical forces at a given point. The usual allowable shearing stress is 10,000#/□″. A size which is safe in flexure is generally satisfactory for shear.

The bearing of each individual member on the pin must be safe. The resisting area for each case is the product of the thickness of the metal in the member and the diameter of the pin. The bearing, as well as the rigidity of the joint, depends upon the "play" of the pin. The holes through which the pin passes must be nearly equal in diameter to that of the pin. For cheap work, $\frac{1}{32}''$ is allowed, but for first-class joints, $\frac{1}{50}''$ is commonly specified. The holes are usually reamed and the pins are turned. The pin is often held in position sidewise by being turned to a smaller diameter at the ends, threaded, and recessed nuts used. Cotter pins are also used to hold the parts in place, but are not advised.

The strength of the eye bars is developed at the pin by **eye-bar heads** which are simply an enlargement of the bar, as illustrated in Fig. 360. These are proportioned by empirical rules, as the stresses are much too complex for precise calculations, and the design is standardized, as there are few manu-

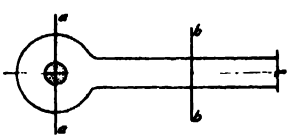


Fig. 360

facturers and most structural shops are not equipped to make them. The head is made stronger than the body of the bar and a common specification is that the net area at plane a–a in Fig. 360 shall not be less than 40% in excess of that at b–b. Eye-bar heads are usually made circular and concentric with the pin, although offset shapes are used.‡

In order to develop the bearing of thin channel webs upon pins, **pin plates** are used. The thickness of the channel is counted upon to resist its value in bearing and the excess is supplied by a pin plate riveted to the channel, as illustrated in Fig. 361. The rivets must be large and numerous enough to develop the bearing carried by the plate (Art. 27). Countersunk rivets are used, if possible, so that the

† A higher working stress is usually allowed on pins for ratios of $\frac{l}{d}$ from 6 to 16, than in usual beams, as the loads are assumed concentrated at the centers of bearing of the members. An allowable value of $40{,}000 - 1000\,\frac{l}{d}$ instead of 24,000 has been suggested (see article in Engineering News Record, Vol. 86, p. 502, by D. B. Steinman).

‡ Refer to "A Study of Stresses in Eye-bar Heads" by J. Balse, Budapest, Hungary, Engineering News Record, Vol. 87, p. 234, for suggested proportions.

parts may be placed closely together. A pin plate may be used on one or both sides of the channel. It should be as wide as the channels will allow in order

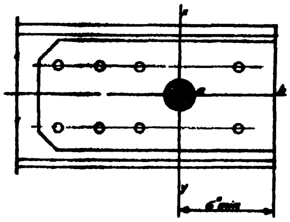


FIG. 361

to obtain sufficient net area, and it should project at least 6″ beyond the center line of the pin. Specifications commonly limit the area xy in Fig. 361 to not less than 40% in excess of the area of the member, and the area ab to not less than 70% of the area xy. The rivets should be in at least two transverse rows and not less than two rivets should be placed beyond the pin hole toward the end of the channel, as shown.

Prob. 227a. If a pin has to carry a load of 64,000#, what size is required if the distance between points of support is 5″? Maximum fiber stress = 24,000#/□″.

Prob. 227b. What is the required thickness of metal in a top chord to give sufficient bearing area to $3\frac{7}{8}$″ pin, having to transmit a stress of 121,400# at an allowable bearing pressure of 24,000#/□″?

Prob. 227c. What size of pin would be required in Fig. 359 if all of the forces were twice those shown? Check the bearing on the pin and design pin plates for the channel if necessary.

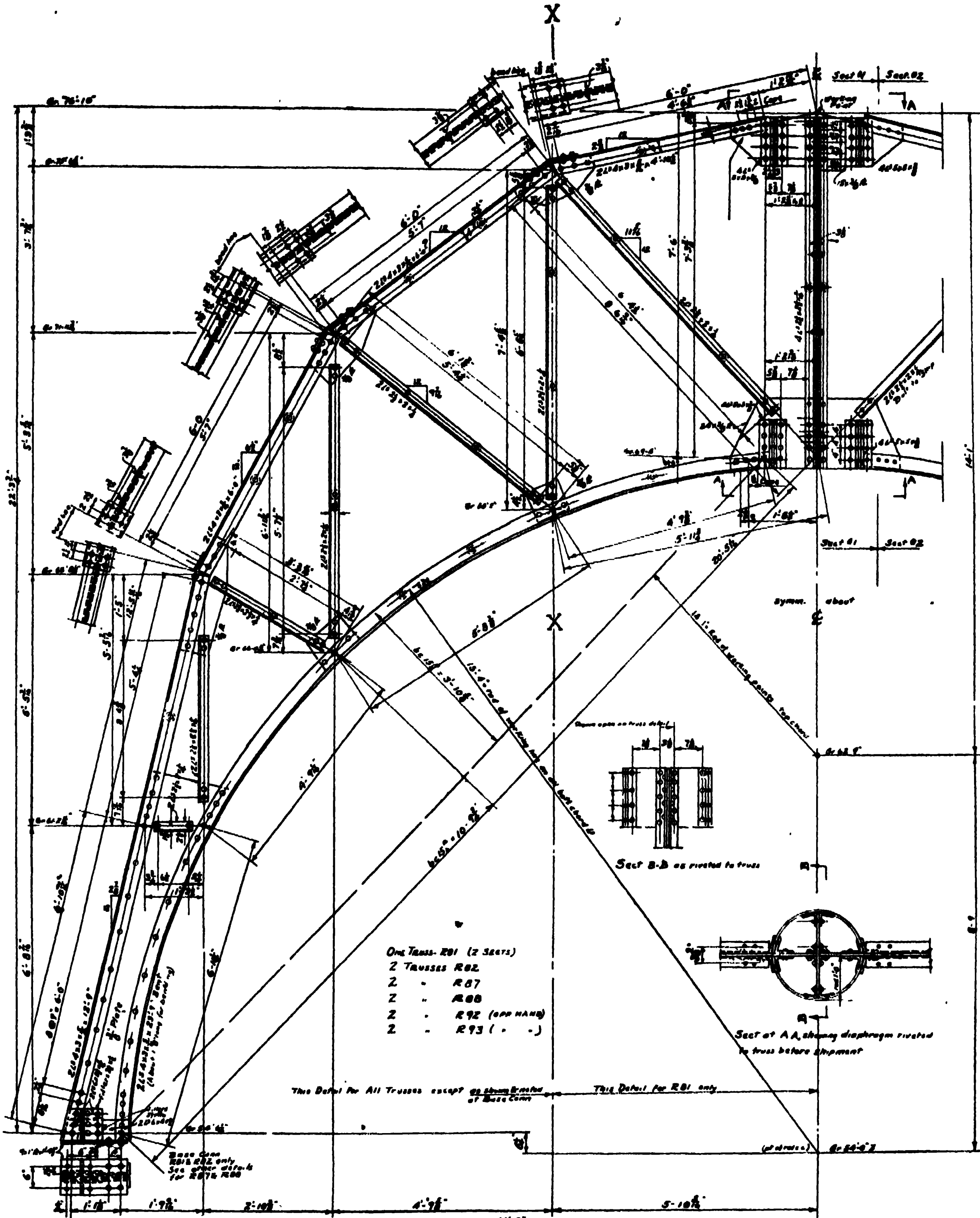


PLATE 30 TYPICAL DETAILS OF A DOME TRUSS

CHAPTER 19

DOMES

228. General.

A dome, in its geometrical conception, is a surface of revolution generated by a line (straight or curved) revolving about a vertical axis. There may be a number of shapes, depending upon the basic line which generates the surface. The following are more common:

(1) Spherical — generated by the arc of a circle,
(2) Conical — generated by a straight line, and
(3) Spheroidal — generated by the arc of an ellipse.

Others may be parabolic in section, or to the form of an Ogee curve. Domes are usually circular in plan, although elliptical plans have been used. The supports are commonly placed at points on a circle, although the supports may be built up from a square. The most common dome is a segment of a sphere.

Domes may be classed as "smooth shell" or ribbed. The former may be considered as superimposed rings of masonry, and these may be of stone, reinforced concrete, or tile. No projections in the form of ribs occur in the usual case. The ribbed dome is made up of a series of individual members, either of wood or steel, but structural steel is more commonly used because of its ease of fabrication and greater stability.

229. Ribbed Domes.

The ribbed or framed dome, which is used quite extensively in modern public buildings, is constructed of ribs (meridians), rings (belts), and diagonals (ties), as shown in Fig. 362 (*a*). The ribs may be single straight members, or they may be half trusses as shown in (*b*). Trusses may be used when the loads are heavy and when a definite roof void is desired. The tendency in recent practice has been to avoid radial trusses, particularly when designing them like arches. In either case the loads act at the joints and are distributed to the rib sections and rings as shown in Fig. 362 (*a*). The number of ribs varies, depending upon the size of the area which is covered, from six to twenty-four being common. A lantern ring is used near the top of the dome to avoid complicated stresses, even if no lantern is contemplated. A wall ring is also usually employed, although it is not theoretically required. The latter opposes the horizontal effect of the forces in the ribs and thus leaves only vertical reactions on the supports.

The forces in a dome are non-coplanar and in order to obtain a simple analysis, components of the forces are taken to reduce them to one plane at a time. Wind loads produce marked effects upon these roof structures as they do upon all exposed roof surfaces. Diagrams or calculations are, therefore, made separately for the dead and live loads and the maximum effect determined for the combination of the individual stresses. The stresses in the rings vary from compression near the top to tension near the base. The maximum tension in an intermediate ring occurs when the dome is fully loaded above it. The maximum compression in an intermediate ring occurs when the dome is fully loaded below that ring. The former condition produces the maximum outward thrust, increasing the tension in the rings and decreasing the compression. The latter condition creates the maximum inward push, which decreases the tension in the rings and increases the compression. A model made of paper will illustrate this action clearly.

The question of live loads is one depending upon assumptions. Snow load could be usually neglected for a pitch exceeding 35° to 40° because the snow would tend to slide off. The wind load depends upon the exposure of the dome, — whether it is above the rest of the structure, or submerged as a part of the low portion of the building, and so on. Normal wind pressures may be assumed from 20 to 30#/□′. In practice, a combined vertical load is usually satisfactory, and 25#/□′ of horizontal projection is a reasonable value. For dead loads, 5#/□′ for framing of the dome, 10#/□′ for tar and gravel or composition roof coverings, and 10#/□′ for plaster, represent average conditions. To this the weights for rafters and sheathing must be added.

The following illustration gives an analytical method of determining the stresses. Analyzing the

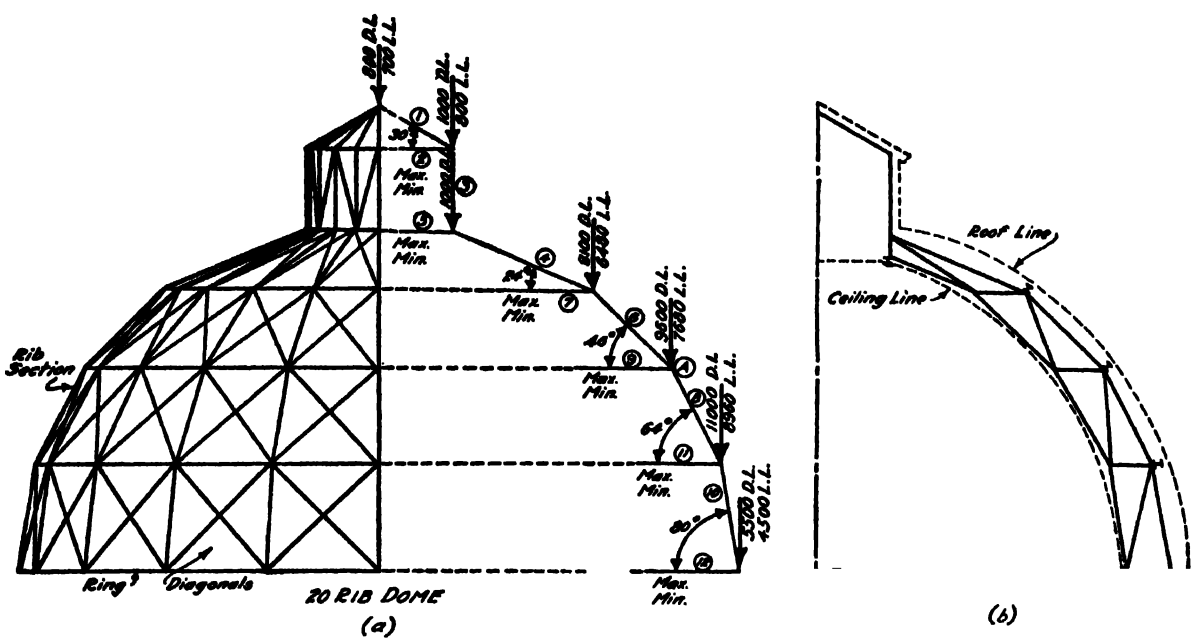


Fig. 362. Ribbed Dome Analysis

(*a*) single frame with rib loads (*b*) truss ribs for dropped ceilings

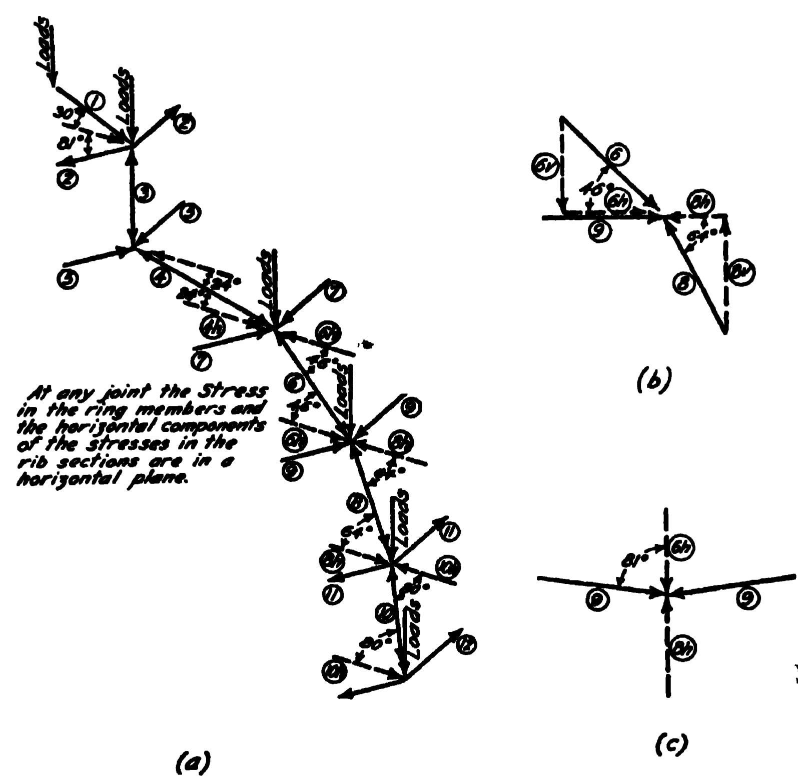


Fig. 363. Analysis of Stresses in Ribbed Domes

(*a*) forces acting upon rib (*b*) and (*c*) resolution of forces at a joint

joint marked A in Fig. 362 (a), two situations arise which give maximum or minimum conditions for the ring stresses:

(1) When the thrust in (6) is assumed as produced by the sum of all of the loads above, and the thrust in (8) is assumed as produced by the same sum plus the dead load of 9600# at the joint.

(2) When the thrust in (6) is assumed as produced by the sum of all the dead loads above, and the thrust in (8) is assumed as produced by the same sum plus the dead and live loads of 9600# and 7680# at the joint.

The maximum stress in any rib section is always determined by the total load above the joint. In Fig. 363 (b) and (c), are shown the forces acting at this joint. The stress in the rib-section (6) is resolved into the force (6 h) and that in section (8) into the force (8 h). The values (6 h) and (8 h) are used to solve (9) as shown in (c). The angle between (6 h) and (9) is, of course, determined by the number of ribs in the dome, — the angle in this case being 81°.

Joint A

Thrust in rib-section (6)

Total load above = 800 + 700 + 1000 + 800 + 1000 + 8100 + 6480 = 18,880

$$\frac{18{,}880}{\sin 46°} = 26{,}200\# \text{ (compression)}$$

Thrust in rib-section (8)

Total load above = 18,880 + 9600 + 7680 = 36,160#

$$\frac{36{,}160}{\sin 64°} = 40{,}300\# \text{ (compression)}$$

Stress in ring members (9)

Maximum Condition

Stress in rib-section (6) = 26,200#

Stress in rib-section (8) $\dfrac{26{,}200 + 9600}{\sin 64°} = 40{,}000\#$

Horizontal component of stress in (6)
= 26,200 · cos 46° = 18,200#

Horizontal component of stress in (8)
= 40,000 · cos 64° = 17,500#

Resultant ring thrust = 18,200 − 17,500 = 700# (tension)

Maximum stress in ring (9) $= \dfrac{700}{2 \cdot \cos 81°} = 2250\#$ (tension)

Minimum Condition

Stress in rib-section (6)

$$= \frac{800 + 1000 + 1000 + 8100}{\sin 46°} \quad 15{,}200\#$$

Stress in rib-section (8)

$$= \frac{15{,}200 + 9600 + 7680}{\sin 64°} \quad 31{,}300\#$$

Horizontal component of stress in (6)
= 15,200 · cos 46° = 10,500#

Horizontal component of stress in (8)
= 31,300 · cos 64° = 13,700#

Resultant ring thrust
= 10,500 − 13,700 = −3200# (compression)

Minimum stress in ring (9)

$$= \frac{-3200}{2 \cdot \cos 81°} = -10{,}300\# \text{ (compression)}$$

The stresses in the diagonals may be obtained by a similar procedure. These are the diagonal components of the maximum difference between the stresses in the rib either side of the diagonal in question. This difference is often determined by assuming one rib fully loaded and the other carrying dead load only. This is a severe and practically impossible condition, but it gives a maximum diagonal stress which provides against unforseen loads.

The lantern ring is subject to axial compression, as well as to some bending produced by thrusts from the ribs. The thrusts are sometimes calculated by assuming the live load to occur on only the two opposite quadrants of the dome. This thrust can then be expressed as so much per linear foot. The bending moment for such a condition is

$$M = \frac{p\ \ }{5}\cdot, \text{ in which}$$

p = the thrust per linear foot, and
r = the radius of the lantern ring, in feet.

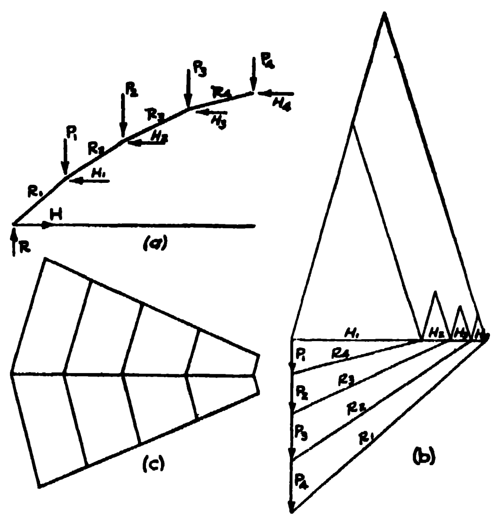


FIG. 364

The ring is often made of a 15″ channel bent to the curve and spliced for its full value. If architectural conditions permit, two diagonal I-beam, interior braces at right angles to each other may be used.

The rib members, in addition to the compression induced by the panel-point loads, must resist any

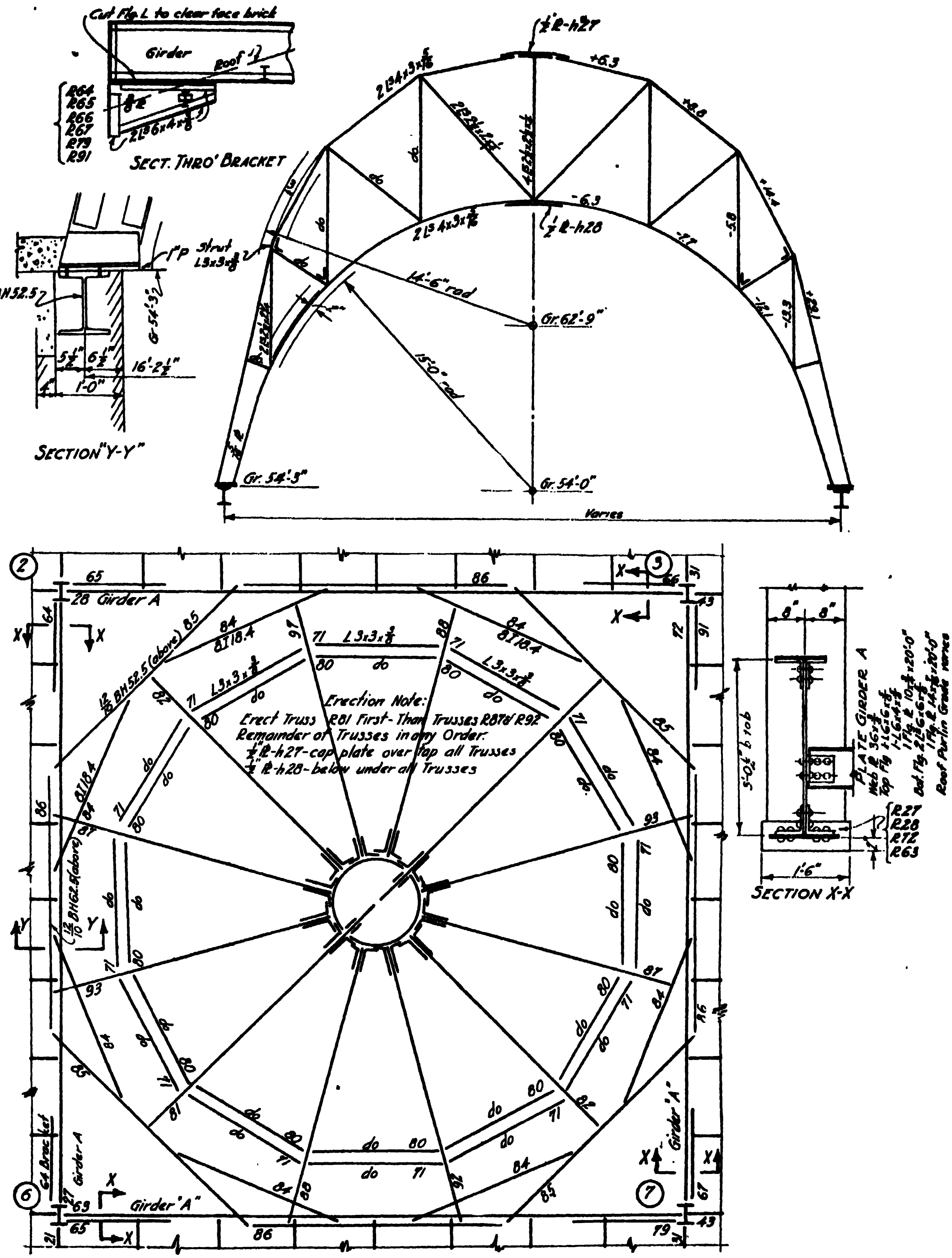


Fig. 365. Plan of Dome Trusses

flexural stress due to their weight and due to intermediate rafter loads.*

A graphical solution of the stresses may be used instead of an analytical one, if desired. Figure 364 illustrates the general method, the diagram in (*a*) showing a rib with its vertical loads. The horizontal forces are used to make all the forces coplanar. The diagram in (*b*) gives the resulting rib stresses, R_1, R_2, etc. The forces H_1, H_2, and so on, may then be resolved into components parallel to the ring members to obtain the stresses in the latter. From these the stresses in the diagonals may be obtained.

230. Trussed Ribbed Domes.

For domes of large diameter, the ribs may be made up of light trusses, as suggested in Fig. 362 (*b*). At the center, near the top, some form of ring is used to frame the upper ends of the trusses into. If a lantern is used, it may be formed by a small circular ring made of a bent angle at the top, and a series of bent T irons to form the curved portion of the lantern roof. These may be collected on a plate and angle drum-member. If the lantern has a vertical portion below, struts may be used to transfer the load down to a lower drum-truss. The dome trusses may then frame between this and the supporting columns or walls, as the case may be. This is illustrated by the line diagram in Fig. 366.

Many domes do not have lanterns of course, and in such cases, a scheme of joining the trusses together at the top must be provided. One method is to make a truss on one of the diameters continuous from one side of the dome to the other, and frame the others into it by means of heavy connection angles and distributing plates, as illustrated in Fig. 365. This gives a main truss with a definite span and acting more directly, like any regular truss, which carries the reactions from the other trusses. Plate 30 shows the details of the trusses indicated in Fig. 365.

* If a member is curved, an additional moment equal to the axial compression times the rise of the curve must be provided for.

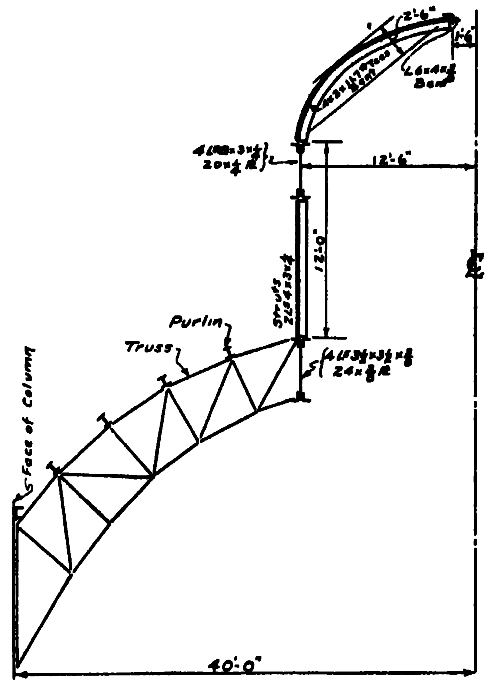


Fig. 366

PART IV

DESIGN OF COLUMNS

CHAPTER 20

GENERAL CONSIDERATIONS

231. General Theory.

The general theory of the design of columns applies with equal accuracy to steel columns, with the necessary corrections which the limitations of this particular material impose. The actual practice of steel column design is discussed in the following articles with the usual assumptions used in the mechanics of column design.

232. Calculation of Loads for Regular Panels.

The calculation of the load carried by a given column is in one sense the most important part of the design. If the load calculated is incorrect, then all subsequent computations are of little value. It is in such work that probably more mistakes are made than in other parts of structural design, and it is wise to train one's self to make load computations with reasonable accuracy. The critical point is to have loads correspond with the actual construction, and not the determination of a load to the nearest pound. It is usually satisfactory to express the load to the nearest 100#. Thus, a load of 116,473# may be called 116,500# for convenience and with sufficient accuracy in results. The column formulas which are subsequently used are not known to be any more accurate.

The calculation of column loads from regular panels of floor framing is a relatively simple matter. The important factors are the number of square feet of **tributary area** and the total load per sq. ft. for which the floor has been designed.* In Fig. 367, the tributary area is indicated by the cross-hatched portion. The bounding lines occur at points halfway between adjacent columns. If the total load per sq. ft. of floor is 212#, the load from the floor proper is (18 × 20) × 212 = 76,200#. To this must be added the weight of the beams and girders which is carried by the column. This is the weight of 3 floor beams (full length) and one girder,† in this case. If the weight of beam "B" is 50#/ft. and of girder "A" is 100#/ft., and the columns are assumed as 12″ on a side, the total from this source is

$$3 \times 50 \times (20 - 1) + 1 \times 100 \times (18 - 1) = 4550\#.$$

When fireproofing, as for steel beams, occurs below the floor proper, the weight should be included in the beam weights per linear foot. Some designers make an allowance of so many #/□′ in the floor load for the weight of the framing. To illustrate, if 12#/□′ had been added into the floor load, the increase would be 12 × (18 × 20) = 4320#, — a reasonable approximation. Procedure of this kind is

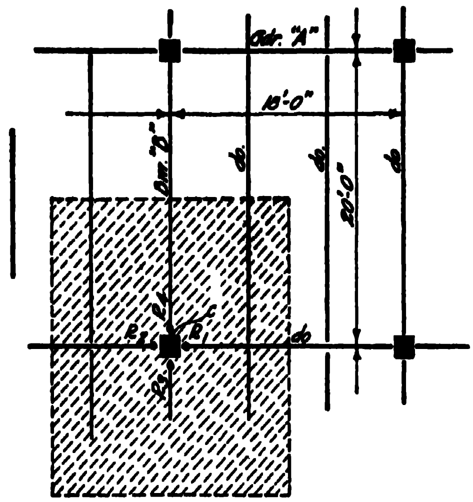


FIG. 367

necessary when the columns are designed independently of the floor calculations, as is sometimes the case on a rush job and when several designers are working at once, or where work is done by the "squad" system. In the majority of instances, however, the floor framing is designed before the columns, so that the actual weights of beams and girders can be used. This procedure is advisable. The above is the common method of determining loads for simple cases. The sum of the reactions R_1, R_2, R_3, and R_4 from the members framing into the column "C" would give the same result. Usually it is more laborious to look up these reactions in the beam computations than to use the tributary area method.

In addition to the load discussed, the load from the column above (if one occurs) must be added. Also, an approximation must be made for the weight of the column itself (and its fireproofing in the case of a steel column, if used). This requires experience and judgment, and only comes with practice in

* Live load requirements and dead loads are discussed completely in Arts. 89 to 91.

† One should make sure that he understands this collective method of obtaining the weights of framing members.

determining how the sizes of columns vary for different loads, and in different stories. If partitions occur within the floor area under consideration, their weight must of course be included. This is sometimes provided for in the load per sq. ft. of floor (Art. 30), or may be included in beam reaction calculations. The total load on any given column may then be summarized (as in this case) as follows:

5th Story col....	78,000 (based upon previous calculation)
5th Floor.......	76,200
Beams.........	4,550
Column........	750 (assumed)
Total........	159,500# — 4th Story Column.

This is the load for which the column is designed.

Prob. 232a. Calculate the column load for a panel similar to Fig. 367, 21'-0" square, if the floor loads are:

Live load — 150#/□', 1" finish flooring — 3#/□', 1" subfloor — 3#/□', 4" cinder concrete — 50#/□', beams 31.8#/ft., and girders 54.7#/ft. Assume 12" BH steel column with 12'-0" story height, column load above, 91,000#.

233. Calculation of Loads for Irregular Panels.

If the floor framing around a column is irregular, the calculation of the column load is usually made with less chance of error, if the beam reactions of the members framing into the column are used, instead of employing a tributary area. This is quite easily done if the beam computations have already been made, as the reactions may be taken from those calculation sheets. Thus in Fig. 368, the load from the floor on column #19 is the sum of R_1 and R_2. If the column load is desired in advance of the beam calculations, the tributary areas of the members framing into the column can be determined, and the proportions of these areas carried to each column. To illustrate in Fig. 368, the area of floor carried by G-16 from the near side is $(16)(18) \div 2 = 144$□'. The center of this area is 8'-0" from column #19. Hence $\frac{8}{16}$ of the load on this area goes to column #27 and $\frac{8}{16}$ of it to column #19, or 72.0□'. This is similar to computing beam reactions. Similarly, on the far side of G-16, the tributary area to the latter is $(10.0)(18) \div 2 = 90$□'. The center of this length is 11'-0" from column #19. Hence $\frac{5}{16}$ of 90□', or 28.1□', goes to this column. In addition the load from G-13 on G-12 is $(8 \div 2) \times (10 \div 2) = 20$□'. The member G-12 brings $\frac{10}{18}$ of this 20□' to G-16 = 11.1□'. Then G-16 carries $\frac{10}{16}$ of 11.1□', or 6.8□', to column #19. The tributary area corresponding to R_1 is then $72.0 + 28.1 + 6.8 = 106.9$□'. With a given total floor load in #/□', the value could be established. Such a unit load would usually include an allowance for the weights of the floor beams and girders, based upon average sizes.

Prob. 233a. Calculate the load from the floor on column #19 in Fig. 368, for the following conditions:

L.L. 150#/□', 1" granolithic — 12#/□', 4" stone concrete slab — 50#/□'. Allow 6#/□' for the weights of beams and girders (average).

Prob. 233b. Using data as in Prob. 233a, calculate the load on column #27 for the floor in Fig. 368.

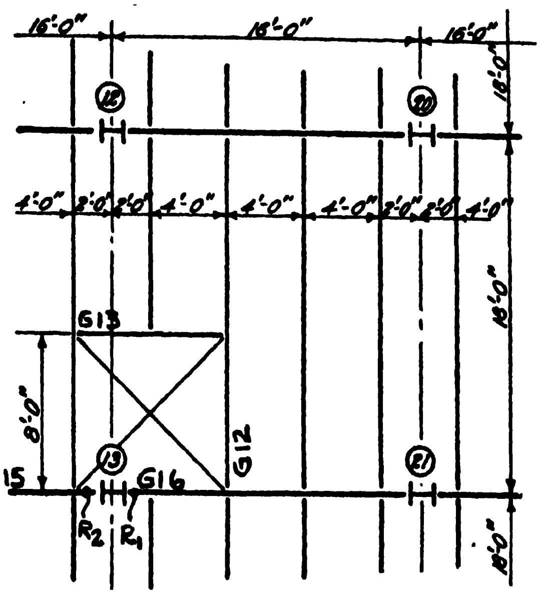


FIG. 368

234. Reduction of Live Load.

It is improbable that the full live load will occur upon a number of floors simultaneously, and in certain instances it may be reduced. The greater number of floors a column supports, the less the average live load per floor will probably be, except in special cases of occupancy.

The question of live load reduction is generally governed by building laws. Certain types of buildings are required to be designed for **full live load.** These are usually **storage buildings, wholesale stores and public garages.** Buildings of two or three stories are generally designed for full live load, and special structures, such as grandstands and the like, naturally should be. For structures in which live load reduction is allowable, the following represents average practice:

SPECIFICATION CLAUSE*

The roof and top floor shall be taken for full live load, and each succeeding lower floor live load may be reduced 5% until 50% of the live load is reached, when such reduced loads shall be used for all the remaining floors.

Other codes vary in their requirements, but the authors recommend the above in the absence of

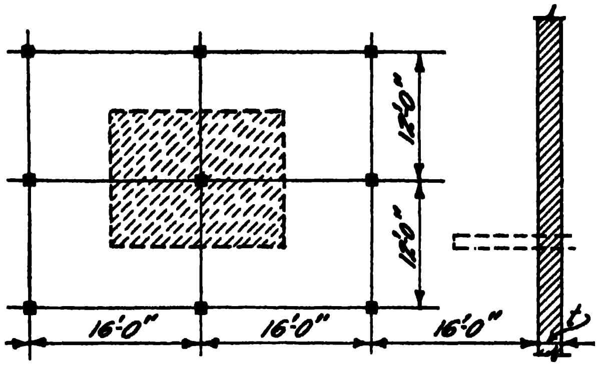
* The Code of Ordinances of the City of New York.

other restrictions, although some authorities claim that this is too conservative. The limits cover the reasonable maximum conditions. If greater loads existed, they would probably be of short duration, and laboratory tests have shown that the usual

14th Story Column — Tributary Area = 400□':

15th Floor L.L. = 400 × 100 = 40,000
16th Floor L.L. = 400 × 95 = 38,000
Roof L.L. = 400 × 40 = 16,000 (No reduction)

L.L. on 14th Story Col. = 94,000#



Floors: L.L. 100#/□'
2" Cinder Concrete fill = 6#/□'
1⅞" Finished Floor = 3#/□'
6" Concrete Slab = 75#/□'
Beams and Girders = 6#/□'

Roof: L.L. = 40#/□'
Tar and Gravel
Roofing 8#/□'
Cinder Fill = 25#/□'
4" Concrete Slab = 50#/□'
Beams and Girders = 4#/□'

For weight of column assume 12 BH Cols.-12'-0" long.

Story / Floor	7	6	5	4	3	2	1	Bas't	Ftg.
Roof	Full L.L. " D.L. Col.	Full L.L. " D.L. Col.	Full L.L. " D.L. Col.	Full L.L. " D.L. Col.	Full L.L. " D.L. Col.	Full L.L. " D.L. Col.	Full L.L. " D.L. Col.	Full L.L. " D.L. Col.	Full L.L. " D.L. Col.
7		Full L.L. " D.L. Col.	85% L.L. Full D.L. Col.	80% L.L. Full D.L. Col.	75% L.L. Full D.L. Col.	70% L.L. Full D.L. Col.	65% L.L. Full D.L. Col.	60% L.L. Full D.L. Col.	60% L.L. Full D.L. Col.
6			Full L.L. " D.L. Col.	85% L.L. Full D.L. Col.	80% L.L. Full D.L. Col.	75% L.L. Full D.L. Col.	70% L.L. Full D.L. Col.	65% L.L. Full D.L. Col.	65% L.L. Full D.L. Col.
5				Full L.L. " D.L. Col.	85% L.L. Full D.L. Col.	80% L.L. Full D.L. Col.	75% L.L. Full D.L. Col.	70% L.L. Full D.L. Col.	70% L.L. Full D.L. Col.
4					Full L.L. " D.L. Col.	85% L.L. Full D.L. Col.	80% L.L. Full D.L. Col.	75% L.L. Full D.L. Col.	75% L.L. Full D.L. Col.
3						Full L.L. " D.L. Col.	85% L.L. Full D.L. Col.	80% L.L. Full D.L. Col.	80% L.L. Full D.L. Col.
2							Full L.L. " D.L. Col.	85% L.L. Full D.L. Col.	85% L.L. Full D.L. Col.
1								Full L.L. " D.L. Col.	Full L.L. " D.L. Col.
Ftg.									
Totals									

Walls:

Brickwork weighs 10# per inch of thickness per square foot of wall.
Assume walls as follows:
(12'-0" stories)
12" brick wall - 6th & 7th stories
16" " " 4th & 5th "
20" " " 2nd & 3rd "
24" " " 1st "
28" Concrete wall Basement

Fig. 369

structural materials can sustain somewhat larger loads temporarily than they can permanently. Also the arbitrary live loads per sq. ft. are, in many city codes, larger than the floor will usually be required to actually carry.

To illustrate the application of the above specification to a 16-story building with panels 20'-0" square and a floor L.L. of 100#/□' and a roof L.L. of 40#/□', the following tabulations are given:

10th Story Column — Tributary Area = 400□':

11th Floor L.L. = 400 × 100 = 40,000
12th Floor L.L. = 400 × 95 = 38,000
13th Floor L.L. = 400 × 90 = 36,000
14th Floor L.L. = 400 × 85 = 34,000
15th Floor L.L. = 400 × 80 = 32,000
16th Floor L.L. = 400 × 75 = 30,000
Roof L.L. = 400 × 40 = 16,000 (No reduction)

L.L. on 10th Story Col. = 228,000#

4th Story Column — Tributary Area = 400□′:

5th Floor L.L.	= 400 × 100 =	40,000		
6th Floor L.L.	= 400 × 95 =	38,000		
7th Floor L.L.	= 400 × 90 =	36,000		
8th Floor L.L.	= 400 × 85 =	34,000		
9th Floor L.L.	= 400 × 80 =	32,000		
10th Floor L.L.	= 400 × 75 =	30,000		
11th Floor L.L.	= 400 × 70 =	28,000		
12th Floor L.L.	= 400 × 65 =	26,000		
13th Floor L.L.	= 400 × 60 =	24,000		
14th Floor L.L.	= 400 × 55 =	22,000		
15th Floor L.L.	= 400 × 50 =	20,000	No further	
16th Floor L.L.	= 400 × 50 =	20,000	reduction	
Roof L.L.	= 400 × 40 =	16,000	(No reduction)	

L.L. on 4th Story Col. = 366,000#

Contrasting code rulings are those of Boston and Chicago. The Boston code prescribes the following reductions of live load:

Carrying one floor	No reduction
two floors	25% "
three "	40% "
four "	50% "
five "	55% "
six floors or more	60% "

The above reductions apply to *all* the floor live load above the column in question. The authors believe that this is too liberal a reduction, particularly for certain types of buildings.* The Chicago code is similar to the New York requirements except that for the succeeding lower floors, after the roof and top floor full live load, the live load is reduced 15% for the next floor and then 5% additional for each succeeding lower floor until 50% of the live load is reached, when such reduced loads must be used for all the remaining floors. It should be noted that in any case the **roof live load is never reduced.**

The following† represents a recent specification in regard to reduction of live loads:

SPECIFICATION CLAUSE

Except in buildings for storage purposes, the following reductions in assumed total floor live loads are permissible in designing all columns, piers or walls, foundations, trusses, and girders:

Reduction of total live loads carried	Per cent
Carrying one floor	0
Carrying two floors	10
Carrying three floors	20
Carrying four floors	30
Carrying five floors	40
Carrying six floors	45
Carrying seven or more floors	50

For determining the area of footings the full dead loads plus the live loads, with reductions figured as permitted above, shall be taken; except that in buildings for human occupancy, a further reduction of one-half the live load as permitted above may be used.

* In the light of the very generous live loads prescribed in the Boston Code, they are probably the same in the ultimate result. It would be far better to adopt uniform provisions both as to loads and reductions.

† Report of Building Code Committee "Minimum Live Loads Allowable for Use in Design of Buildings," U. S. Dept. of Commerce.

This is suggested because the committee considers the usual requirement too conservative, based upon the evidence received in their study.

Prob. 234a. Refer to the data given in Prob. 232a, and assuming that the construction is the same at each floor, calculate the typical column loads for each story of a seven story building and basement in accordance with the New York Code. Assume all columns the same weight for simplicity.

Prob. 234b. Repeat Prob. 234a for the Boston Building Code.

235. Tabulation of Loads.

It is usually wise for a designer to tabulate column loads when there are a considerable number of stories and a large number of columns. This

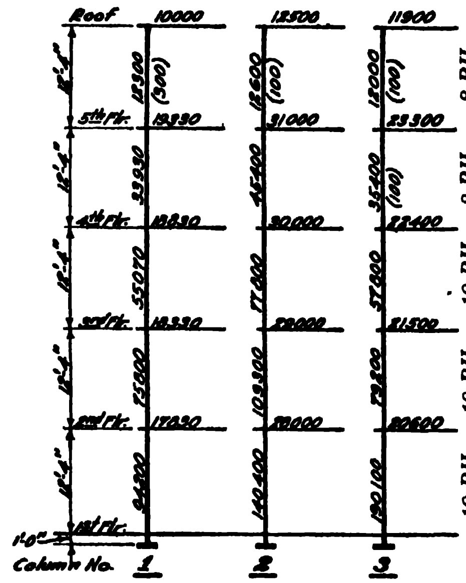


Fig. 370

helps to provide a compact set of values when the design is undertaken and readily lends itself to checking. Figure 369 illustrates one form of tabulation which is convenient, and Fig. 370 another type.

Prob. 235a. Calculate the loads for the typical column stack illustrated in Fig. 369.

236. Wall Column Loads.

Wall columns must not only carry the portion of floor or roof tributary to them, but also the weights of the wall brought to them by the spandrel beams. Such weights include the wall proper, sills, sash, lintels, belt courses, cornices, and so on, as the case may be.

The loads from the floor or roof are naturally the proportionate parts of the tributary floor areas adjacent to the columns. Thus in Fig. 371 (*a*), the

floor area tributary to column No. 2 is one-half a panel load, as indicated by the cross-hatched portion, while column No. 1 receives only one-quarter of the typical panel load, as indicated. To these loads must be added the weights of the wall construction which are carried to each respective column by the wall framing. Hence a designer must be thoroughly familiar with the approximate weights of the materials that enter into such construction. Table 75 will be useful in this work.

TABLE 75

WEIGHTS OF WALL MATERIALS

Architectural terra cotta, voids unfilled	72#/c.f.
" " " " filled	120#/c.f.
Brick, common	120#/c.f.
pressed	130#/c.f.
Concrete, cinder	108#/c.f.
stone	150#/c f.
Concrete block (considering voids)	100#/c.f.
Granite	170#/c.f.
Gypsum blocks, 2″ solid	7#/□′
3″ hollow	10#/□′
3′ solid	12#/□′
4′ hollow	13#/□′
5′	15#/□′
6′	16#/□′
8′	22#/□′
Limestone	160#/c.f.
Marble	170#/c.f.
Plaster	5#/□′
Sandstone	145#/c.f.
Sash, wood	8#/□′
steel	10#/□′
Split terra cotta furring (2″)	9#/□′
Terra cotta blocks, 2″	14#/□′
3″	16#/□′
4″	18#/□′
5″	21#/□′
6″	24#/□′
7″	29#/□′
8″	32#/□′
9″	36#/□′
10″	38#/□′
12″	42#/□′
Wood strapping and lath	5#/□′

Illustrative Prob. 236a. Calculate the typical load on column No. 2 in Fig. 371 for the following data:

L.L. = 100#/□′. ⅞″ maple finish floor. ⅞″ sub floor (wood). 3″ cinder fill and screeds. 7″ terra cotta arches. Plastered ceiling 8#/□′. Plaster, lath and strapping on walls 10#/□′. 12″ brick curtain walls. Allow 15#/□′ of floor for random partitions. Sash and frame 8#/□′. Terra cotta frieze, 90#/linear ft. Window sills, cut stone, 95#/linear ft.

Floor

L.L. =	100#/□′		
Fin. Flr. =	3	Tributary area	= 15 × 7
Sub. Flr. =	3		= 105□′
3″ Cinder fill =	27	Load from floor	= 105 × 165
7″ T.C. =	24		= 17,300#
Ceiling =	8	Floor bms.	
	——	9 I 21 × 7.0′ × 3 =	440
T.L. =	165#/□′	Spandrels	
		15 I 42 × 15 =	630
		Total =	18,370#

Wall

Brick pilaster 6′-0″ × 1′-8″, 12′-0″ (e)).
Brickwork 6.0 × 1.67 × 12 × 120
Plaster, lath, and strapping
6.0 × 12 × 10#
T.C. Facia 90#/ft. × 9.0
Sash 7′-4″ high
7.33 × 8# × 9.0 = 530
Brick curtain between pilaster 12″ thick
height above bm. to sill = 4″ + (2′-8″) − 6″ = 2′-6″
Volume = 2.5 × 1.0 × 9.0 = 22.5 cu. ft.
Deduct for panel, 4″ inset, 1′-4″ high, 7′-8″ long = 0.33 × 1.33 × 7.67 = 3.4 cu. ft.
Brickwork, net, = (22.5 − 3.4) 120#/cu. ft. = 2290
Plaster, lath, and strapping,
[(2′-8″) − 6″] × 9.0 × 10# = 180
Window sill, 95#/ft. × 9.0 = 860
Total = 19,790#

Assume column weighs 80#/linear ft.

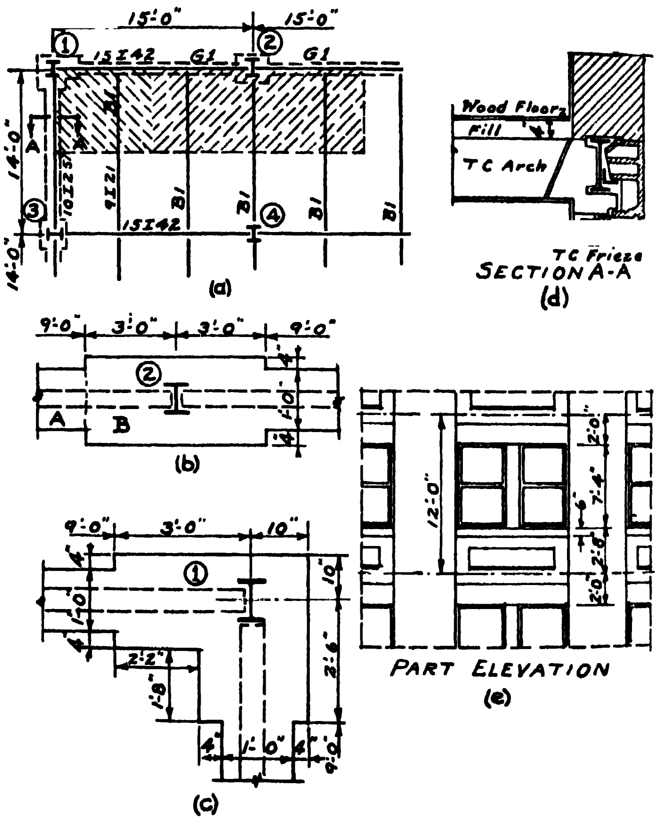


FIG. 371

Summary

Floor load	18,370
Wall load	19,790
Column, 80 × 12	960
Total	39,120#, say 39,200#

Notes. In the above result, no reduction of live load has been considered (Art. 234). When calculating the weights

of the I-beams, the designer may not know their sizes at the time the column load is being computed. In such a case, for the steel floor framing may be made by adding 5# to 10#/□′ to the unit floor load, depending upon the sizes of the beams. Here, an allowance of 8#/□′ would be liberal.

If the spandrel beams and girders have been designed before the column loads are calculated, time may be saved by adding beam reactions. Thus in Fig. 371 (*a*), the load on column No. 2 at that level is 2 reactions from G1, plus the reaction from B1.

Prob. 236b. Calculate the typical load on column No. 3 in Fig. 371. Plan-sections and elevation similar to those shown in the figure, except side pilasters in 14′-0″ direction are 5′-0″ wide (9′-0″ constant sash dimension).

Prob. 236c. Calculate the typical floor load on column No. 1 in Fig. 371.

Prob. 236d. Calculate the load at the roof level for column No. 2 in Fig. 371 (*a*) for the following conditions:

5 ply tar and gravel roofing, 8#/□′, L.L. = 40#/□′.
6″ T.C. arches. 3″ cinder fill. Suspended ceiling 15#/□′.
8 I 18 roof beams, 12 I 31.8 spandrel girders and beams.

Allow 1040#/linear ft. for the cornice and top story window head arrangement. Columns 8 H 32.

Prob. 236e. Calculate the typical load at the floor level on column No. 4 in Fig. 371 (*a*). See Illustrative Prob. 236a for data. Assume column fireproofing 12″ × 12″, of cinder concrete.

Prob. 236f. Calculate the typical roof load on column No. 4 in Fig. 371 (*a*). Assume column fireproofing 12″ × 12″, of cinder concrete. See Prob. 236d for data.

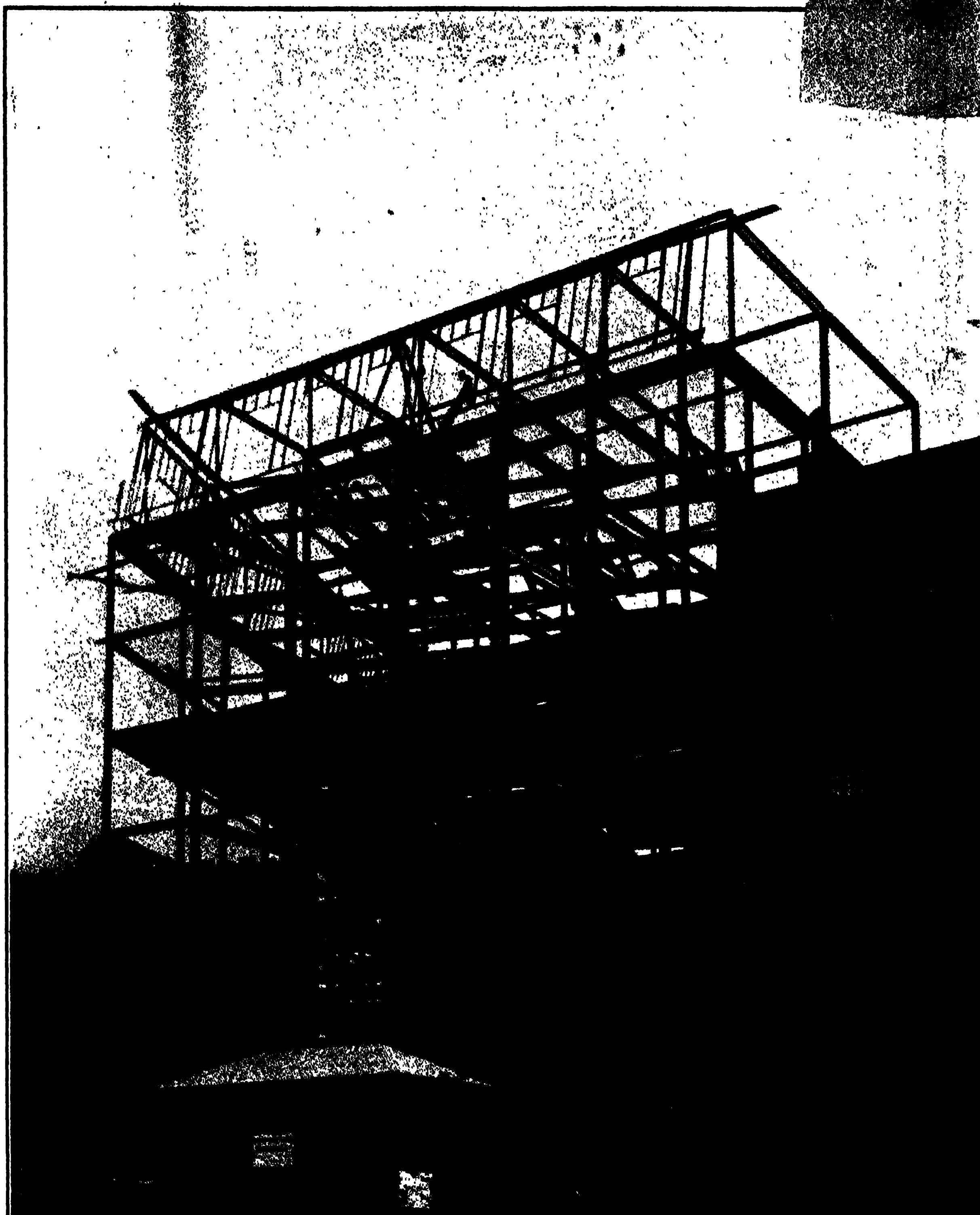


PLATE 31 TYPICAL STEEL COLUMNS

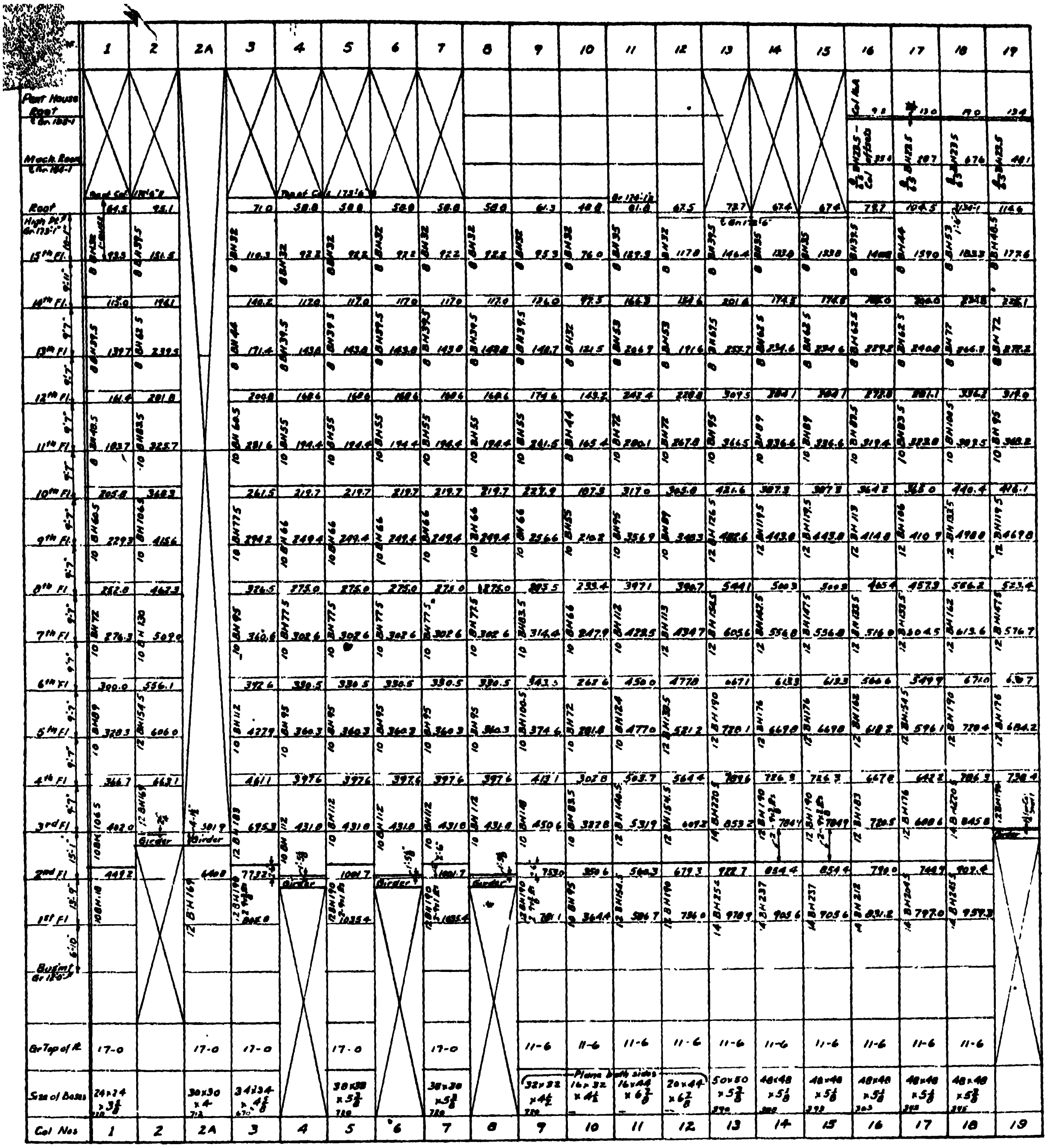


PLATE 32 TYPICAL COLUMN SCHEDULE

CHAPTER 21

STEEL COLUMNS

237. General Considerations for Column Sections.

Steel columns are probably the most used type of column in building construction as a whole, as they may be made to carry heavy loads with relatively small sectional areas compared with other materials.

There are a number of features which should be considered in the make-up of the cross-section of a structural steel column. On an ideal basis, the metal should be well away from the neutral axis in order to provide as large a radius of gyration as is reasonably possible for a given sectional area. Theoretically, the hollow circle is ideal, but is not practicable for rolled shapes. There should be a balanced distribution of metal, that is, the webs, flanges, and cover plates should be of nearly the same thickness if possible. Maximum thicknesses are governed by the punching for rivet holes, unless drilling is to be resorted to. The cheapest column is not necessarily the one of least weight but the one which costs the least in fabrication and erection. The arrangement of the component parts should be simple, as less secondary stress is developed. The closed column is actually stronger than the open one on account of the friction of the parts in contact. Some of the considerations which are important in the selection of a column type are:

(1) cost of fabrication,
(2) cost of erection,
(3) loads to be as nearly concentric as possible,
(4) possibility of good beam connections and splices,
(5) possibility of changing thicknesses of metal from one tier to the next, and the provision of direct bearing,
(6) accessibility for field painting and inspection,
(7) number of rivets to be driven and ease of driving them,
(8) ease of encasement in fire-resisting materials, and
(9) available space for pipes and conduits.

238. Typical Sections.

Figure 372 shows a large variety of cross-sections for structural steel columns. Because of the types of structural shapes, an unlimited number of sections may be devised, but for the reasons given in Art. 237, only a few typical sections are commonly used. (Angle struts are discussed in Sect. 23A). The cross-section shown in Fig. 372 (*f*) is sometimes used for long columns carrying light loads, and is a good form if encased in concrete, which materially stiffens it.

Plate and angle columns (Art. 244) are one of the most common types of fabricated columns, particularly (*h*) and (*i*) in Fig. 372. These are relatively cheap, easily fabricated, allow for the provision of good connections, and are accessible for field painting. Type (*g*) possesses little merit compared with (*h*), and types (*k*) and (*l*) cannot be wholly field painted, and type (*l*) involves difficult fabrication. Type (*m*) is sometimes used in railroad work.

H columns are the most commonly employed of all the types (Art. 246), and usually without cover plates. Channel columns are occasionally used for moderate loads. They provide relatively large radii of gyration for given areas. The distance, *d*, should be made such that the radii of gyration about the two rectangular axes are as nearly equal as possible. The type (*r*) in Fig. 372, sometimes called a " box " column, is theoretically more economical of material than type (*q*), the latticed column (Art. 248), and is more rigid. Types (*s*) and (*t*) provide poor beam connections, and (*x*) involves difficult fabrication. Type (*q*) is satisfactory for light loads, while (*r*) and (*u*) have a good distribution of metal and are used for heavier loads. The plates, *A*, are added only when heavy loads are to be carried.

I-beam columns are not very economical, as they are relatively weak about the 2-2 axis, as indicated in Fig. 372 (*y*), and hence are not commonly employed. They are used as outside columns in light mill buildings (Chap. 30), and lend themselves to concealment in thin partitions when this is a necessity. **Z-bar columns are rarely used in modern practice** and are uneconomical, as considerable metal is located near the neutral axis and although places for connections for beams off the center line of the column are available, these are badly eccentric. Careful riveting is required, field painting inside the types (*dd*) and (*ee*) is impossible, and poor beam connections almost alwaysr esult. Type (*ff*) is too complicated for ordinary use. Figure 372 shows some of these older types principally to illustrate relative dis-

advantages as compared with modern sections, and to show what has been used, at times, in the past.

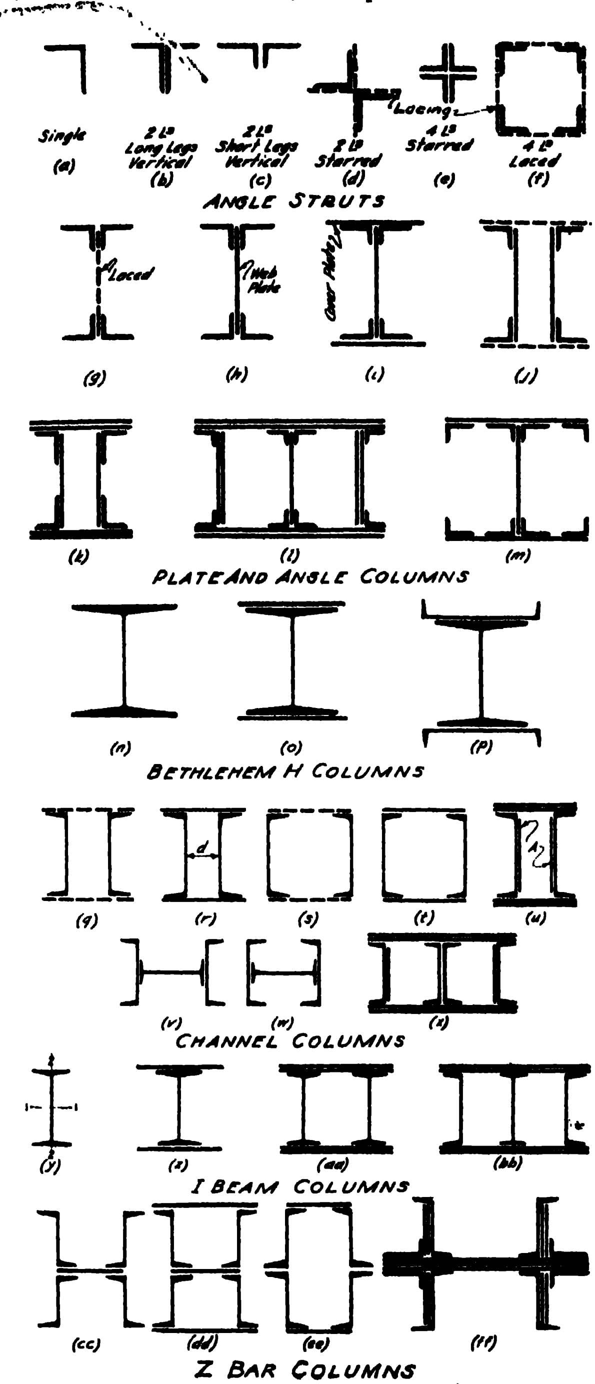


FIG. 372

239. Patented Sections.*

Special sections similar to those illustrated in Fig. 373 were formerly employed from time to time. The Gray column gives a large radius of gyration for the minimum amount of metal, and is particularly adaptable when encased in concrete. It is in reality four pairs of double angle struts tied together by battens. Good connections are possible and each load is reasonably concentric on each strut, although the battens do not thoroughly equalize the loads. The sizes range from 10″ □ minimum to 20″ □ maximum, in patented arrangements. The chief objection to this type is its high cost of manufacture. Figure 373 shows the Keystone octagonal, the Larimer, and the Phoenix types. While these offer good distribution of metal, they are objectionable because of the difficulty of framing in the beams and field painting. For these reasons, **patented types are seldom used in modern practice.**

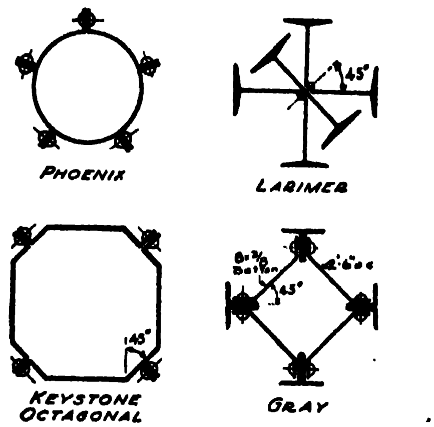


FIG. 373

240. Fire Protection.†

SPECIFICATION CLAUSES‡

The protection shall cover the interior columns at all points to a thickness of not less than 3 inches and be continuous from the base to the top of the column. The extreme outer edges of lugs, brackets, and similar supporting metal may project to within 1 inch of the outer surface of the protection.

NOTE.—Much has yet to be learned regarding the necessary thickness and methods of application of different column coverings to produce efficient fire protection. The investigations on this subject now in progress at the Underwriters' Laboratories in Chicago, will furnish much needed information. When secured, it may justify changes in the requirements herein made.

If brick or blocks are used for fireproofing columns, they shall be accurately fitted, laid with broken joints, and all spaces between the outside layer and the metal solidly filled with masonry; or a concrete filling may be used. No voids between the metal and the protecting casing shall be permitted.

Galvanized steel wire not smaller than No. 12 gauge, shall be securely wrapped around block column coverings so that every block is crossed at least once by a wire. The wire shall not be wound spirally around the column, but each

* These sections are discussed principally to show the progress made in structural steel work, and also to illustrate disadvantages as compared with modern sections.

† For specifications regarding fire protection of exterior columns, see Art. 236.

‡ From the Building Code of The National Board of Fire Underwriters, New York City.

turn or bend shall be a separate unit and shall be twisted tightly or otherwise securely bound. Other equivalent anchorage may be employed if approved by the Superintendent. No block used for this purpose shall exceed 12 inches in vertical dimension.

NOTE.— Any method which would securely lock the blocks in place, or hold them by substantial interior metal ties, would be superior to the wire wrapping above described.

Columns located in damp places shall receive a coat of at least 1 inch of Portland cement mortar before application of the fireproofing.

Columns made of steel or wrought iron pipe filled with concrete, shall be protected by at least 1½ inches of fireproofing.

Where the fireproofing of columns is exposed to damage from trucking or handling of merchandise, the fireproofing shall be jacketed on the outside for a height of not less than 3 feet from the floor with metal or other approved covering.

FIG. 374*

Experience has demonstrated the necessity of thoroughly protecting steel columns supporting walls or floors against fire. In first-class construction, this must be done to conform with specifications. An unprotected column is dangerous in a fire, and while not combustible like timber, it buckles due to the action of the heat, and may even cause failure more readily than some other material (Fig. 374). In second-class construction, — where the frame itself is not planned to be especially fire-resisting — steel columns are sometimes left exposed, or simply covered with materials for the sake of appearance. The reason is of course to keep first costs low.† In structures such as mill buildings, the columns are usually left uncovered.

* Courtesy of Inspection Department, Associated Factory Mutual Insurance Companies.
† This is very poor ultimate economy.

Where columns are to be encased the following materials are used as:

(a) brick,
(b) terra cotta, or concrete tile
(c) cinder or stone concrete, or
(d) metal lath and cement plaster.

The choice of the material will be determined by the kind used near the columns and thus one finds brick used to encase columns which form the pilasters of brick walls, either outside or division; terra cotta or concrete tile, where abutting partitions or "back-up" is of these materials; and when the floor construction is a cinder or stone concrete slab the most economical fireproofing is concrete. From the standpoint of fire resistance, cinder concrete is by far the most efficient material. Any of the above

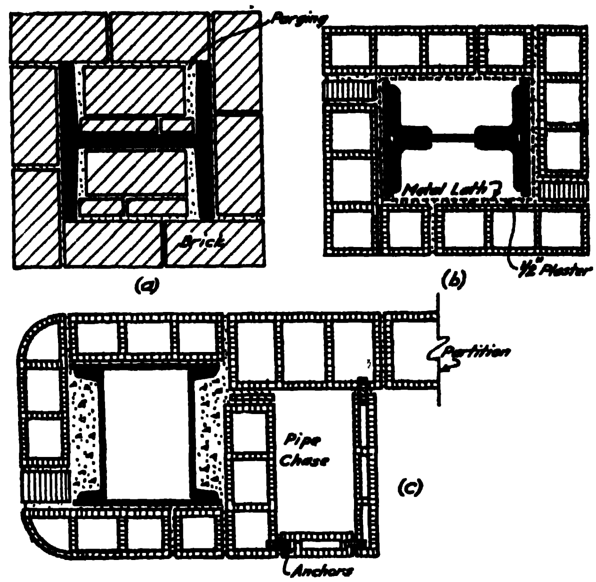


FIG. 375. BRICK AND TILE ENCASED STEEL COLUMNS

(a) brick encased Bethlehem column — steel parged 18″ to 24″ in advance of brick.
(b) tile enclosed plate and angle column — square corners
(c) tile encased plate and channel column — rounded corner — pipe chase and intersecting partition

materials are good and the deciding factors may be price, architectural detail, or both, although it may be said that concrete is most used in general. When terra cotta or brick are used they are often set with broken joints around the columns, as illustrated in Fig. 375.

Any piping carried alongside of the columns should be provided for in a separate chase, built outside of the main column covering, as shown. The spaces between the hollow tile and the steel should be filled with concrete or cement mortar to prevent the possibility of corrosion and for addi-

tional security against fire. A low rate of insurance may be secured when the columns are properly protected, for the durability of the entire building is dependent upon the integrity of the columns.

SPECIFICATION CLAUSES

Free-Standing Columns All free-standing and other columns, not enclosed by brick work, shall be given a coat of Portland cement mortar, ½" thick, on all sides, after which they shall be enclosed in 4" blocks with the corners bonded.

All voids shall be filled with mortar and tight joints shall be formed at the intersections of the blocks with the under side of the concrete floor slabs.

While the treatment of the surfacing of the column is largely the province of the architect, the matter of how the fire protection shall be provided, and so on, is often decided by the engineer. In any case, **the weight* of such protection must be included in the column loads.**

Prob. 240a. Calculate the weight of the column and fire protection in Fig. 375 if the section is a 10BH71.

241. Column Formulas.

There are a number of reliable formulas for designing structural steel columns. Of these, the most commonly used in actual design practice are of the "straight line" type, because they are simpler to apply and they are sufficiently accurate. One of the most usual types is

$$p = 16{,}000 - 70\frac{l}{r} \qquad (S\text{–}53)$$

in which

p = the maximum allowable compressive stress in #/□",

l = the unsupported length of the column in inches, and

r = the minimum radius of gyration of the cross-section, in inches.

This is the formula of the New York City Building Code, and formerly used by the American Railway Engineering Association. The latter ruling limited the maximum stress in any case to 14,000#/□".

In any actual case in practice, the formula specified by the governing ordinance must of course be used. In the absence of any restrictions, the authors recommend the above formula (S-53). From the graph in Fig. 376, it is obvious that this formula represents a fairly good average of those plotted, and leans somewhat toward more conservative stresses. Other straight line formulas are:

$$p = 19{,}000 - 100\frac{l}{r} \quad \text{(American Bridge Company) (maximum 13,000\#/□")} \qquad (S\text{–}54)$$

$$p = 15{,}000 - 50\frac{l}{r} \quad \text{(new American Railway Engineering Association) (maximum 12,500\#/□")} \qquad (S\text{–}55)$$

$$p = 20{,}000 - 100\frac{l}{r} \quad \text{(American Society of Civil Engineers)} \qquad (S\text{–}56)$$

(For values $\frac{l}{r}$ 80 to 120, below $\frac{l}{r} = 80$, $p = 12{,}000$#/□")

$$p = 15{,}200 - 58\frac{l}{r}† \quad \text{(old New York City code)} \qquad (S\text{–}57)$$

$$p = 15{,}000 - 40\frac{l}{r} \quad \text{(Syracuse Code).} \qquad (S\text{–}58)$$

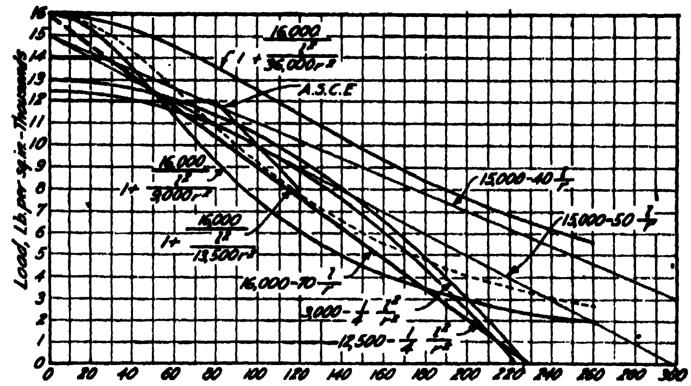


Fig. 376

The usual specifications limit the ratio of slenderness, $\frac{l}{r}$, for columns in order not to allow extremely long and thin members, which might be especially dangerous if larger loads than were anticipated had to be carried for a short time. Figure 377 shows

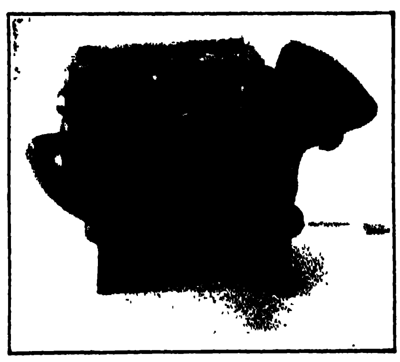


Fig. 377

a portion of a column failure in a laboratory test, which illustrates a typical failure in an extreme condition. For lesser loads, the tendency toward similar failures is present and must be guarded

* Refer to Art. 236 for the weights of materials.

† Based on the similarity of the column flexure curve to a sine curve. A factor of 0.8 was used, or 0.8 × 70 = 56 + (58 used), comparing favorably with formula S-53.

against. A relatively small deformation in a column may produce stresses in other portions of the frame which may become serious.

SPECIFICATION CLAUSE

The effective or unsupported length of main compression members shall not exceed 120 times, and for secondary members 200 times the least radius of gyration.

Such recommendations are for steady stresses. Secondary members are those in wind bracing and the like. Some engineers make a practice of varying the limitations of the ratio of slenderness according to the particular members, such as:

	Max. $l \div r$
Crane columns	100
Heavy loads	125
Shed columns	150
Truss chords	150
Truss webs	175
Wind struts	200

Instead of straight line formulas, some codes and specifications give those of the second degree (Art. 119, Book I).* The following are representative:

$$p = \frac{12,500}{1 + \frac{l^2}{36,000\, r^2}} \text{ for square ends (Cambria Steel Co.)} \qquad (S\text{–}59)$$

$$p = \frac{12,500}{1 + \frac{l^2}{18,000\, r^2}} \text{ for pin ends (Cambria Steel Co.)} \qquad (S\text{–}60)$$

$$p = \frac{14,500}{1 + \frac{l^2}{11,000\, r^2}} \text{ (Philadelphia Code)} \qquad (S\text{–}61)$$

$$p = \frac{16,000}{1 + \frac{l^2}{20,000\, r^2}} \text{ (Boston Building Law)} \qquad (S\text{–}62)$$

These are based in many cases upon the Rankine (or Gordon) formula (Art. 119, Book I). The coefficient in the denominator of the first two formulas is smaller than Rankine recommended but this is more or less compensated for by the conservative value of 12,500#/□″ as a maximum. These formulas are very satisfactory, but require more time for calculations.

242. Design of Columns in General.

The design of structural steel columns must be done by more or less "cut and try" methods, as the allowable stresses vary with the section, and the section required depends upon the load to be carried. One method or another of assuming a trial section must be used, and this section must be checked up. A valuable reference in work of this kind is the tabulation of approximate radii of gyration of various sections, shown in Fig. 378. The various handbooks are of great value also, in determining sections, with their tables of areas, radii of gyration, safe loads, and so on.

Illustrative Prob. 242a. Design a column similar to type 20, Fig. 378, to carry a load of 285,000#. Length = 12′-8″. Assume 12″ plates ($h = 12\frac{1}{2}$″).

From Fig. 378, approximate $r = 0.36\, h = 0.36 \times 12.5 = 4.5$″.

$$\text{Trial stress } p = 16,000 - 70\,\frac{l}{r}$$

$$= 16,000 - \frac{70 \times 152}{4.5} = 13,640\#/\square''$$

$$\text{Trial area} = \frac{285,000}{13,640} = 20.9\square''$$

$$\begin{aligned} \text{Try} \quad 4 \llcorner^{s}\ 4 \times 3 \times \tfrac{7}{16} &= 11.48\square'' \\ 2 \text{ Pl. } 12 \times \tfrac{7}{16} &= \underline{10.50} \\ &\ 21.98\square'' \end{aligned}$$

$$I \text{ of Pls.} = 2 \times \frac{0.43 \times (12)^3}{16} = 125.5$$

$$\begin{aligned} \llcorner^{s} &= 4 \times 2.18 = 8.7 \\ A \cdot d^2 \text{ of } \llcorner^{s} &= 11.48 \times (5.2)^2 = \underline{310.3} \\ I \text{ (total)} &= 444.5''^4 \end{aligned}$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{444.5}{21.98}} = 4.48''$$

$$p = 16,000 - \frac{70 \times 152}{4.48} = 13,620\#/\square''$$

$$\left.\begin{aligned} A &= \frac{285,000}{13,620} = 21.0\square'' \text{ required} \\ A &= 21.98\square'' \text{ actual} \end{aligned}\right\} \text{O.K.}$$

Use $4 \llcorner^{s}\ 4 \times 3 \times \frac{7}{16}$
$2 \text{ Pl. } 12 \times \frac{7}{16}$

The distance, b, in type 20, Fig. 378, must be such that the value of the radius of gyration about the axis $y - y$ (r_{y-y}) is not less than the value of r_{x-x}. Thus

$$r_{y-y} = 0.53\, b, \text{ or}$$
$$4.48 = 0.53\, b, \text{ and } b = 8.46'' \text{ minimum.}$$

The value of b would be made some multiple of $\frac{1}{4}$″ usually, — in this case, say $8\frac{1}{2}$″. In general, an attempt should be made to obtain a "balanced" distribution of metal. The ideal section is that in which the radii of gyration are equal about each axis. Practical considerations alter this to some extent, of course. The dimensions of the column sections above are an influencing factor. It is also desirable to have the column section nearly square.

Illustrative Prob. 242b. Design a column similar to type 15, Fig. 378, to carry a load of 114,000#. Length = 14′-6″. Assume outstanding legs of $\llcorner^{s}$ are 5″.

Then $b = 5'' + 5'' +$ (thickness of lacing, say $\frac{1}{4}$″) = 10.25″

$r_{x-x} = 0.42\, h$ and $r_{y-y} = 0.215\, b$.

Assume h is made such that r_{y-y} controls,

$r_{y-y} = 0.215 \times 10.25 = 2.21$″ approximately

$$\text{Trial stress} = 16,000 - \frac{70 \times 174}{2.21} = 10,470\#/\square''$$

$$\text{Trial area} = \frac{114,000}{10,470} = 10.87\square''$$

* There has been some discussion in recent years relative to having engineers lean more to the use of formulas of this type, as they probably do give results nearer to actual conditions than straight line formulas (see Engineering News Record, March 10, 1921, and November 2, 1911).

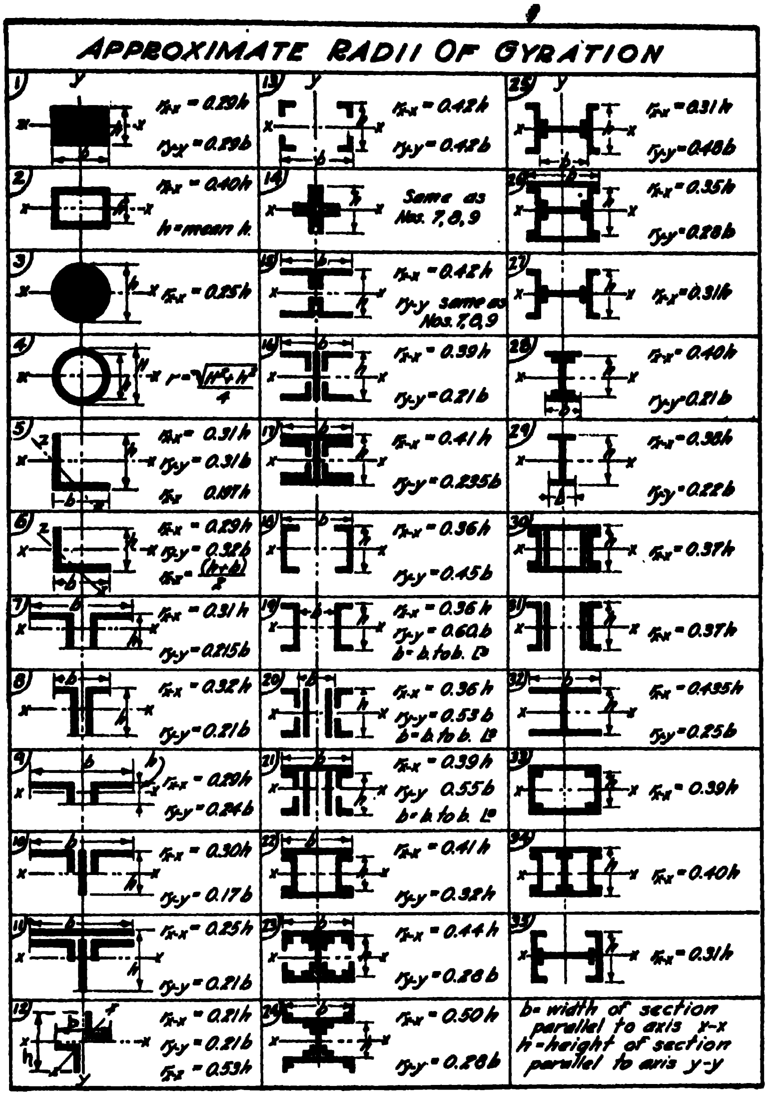


FIG. 378

$$\text{Trial area of } 1 \text{ ⅃} = \frac{10.87}{4} = 2.72\square''$$

Try 4 ⅃ $5 \times 3 \times \frac{3}{8} = 4 \times 2.86 = 11.44\square''$

I of 4 ⅃ $= 4 \times 7.4 = 29.6$

$A \cdot d^2 = 11.44 \times (1.8)^2 = 37.1$

I (total) $= 66.7''^4$

$$r = \sqrt{\frac{66.7}{11.44}} = 2.42''$$

$$p = 16{,}000 - \frac{70 \times 174}{2.42} = 10{,}970\#/\square'' \text{ allowable}$$

$$\frac{P}{A} = \frac{114{,}000}{11.44} = 9950\#/\square'' \text{ actual. O.K.}$$

Use 4 ⅃ $5 \times 3 \times \frac{3}{8}$.

$r_{x-x} = 0.42\,h = 2.42 = 0.42\,h$, or $h = 5.76''$ min.

h is usually made 12″ minimum on account of lacing details.

Prob. 242c. Design a column similar to type 19, Fig. 378 to carry a load of 118,000#. Length = 13′-9″. Try 10″ channels.

Prob. 242d. Design a column similar to type 16, Fig. 378, to carry a load of 450,000#. Length = 15′-0″. Assume cover plates 14″ wide, 6″ outstanding legs of angles, and 12″ web plate (12½″ back to back of angles).

Prob. 242e. If a column section is similar to type 22, Fig. 378, and is composed of two 10][20 and two 12 × ½ Pls., what safe load can it sustain, if the length is 15′-6″? Use $p = 15{,}000 - 50\,\frac{l}{r}$.

243. Eccentrically Loaded Columns.

The foregoing discussion has been based upon the assumption that the loads are applied at the longitudinal center of gravity axes of the columns (concentric loads). Such a condition is theoretical only, and cannot be obtained when practical details

are considered. **However, for ordinary cases of beams framing into columns, the loads are considered as concentric.** For designs in which live loads of 150#/□′ or less are involved, eccentricities are generally neglected. For larger live loads, such as in warehouse construction, eccentricities due to beam reactions should be considered.

When a beam frames into the web of a typical plate and angle, or Bethlehem, column, the beam reaction is applied quite near to the axis of the column.* When a beam frames into the flange of a column, the distance to the point of application of the reaction is obviously a considerable distance from the axis of the column. If there are beams on the two opposite faces of the column, and both beams are alike and fully loaded, the bending caused by each reaction would balance that of the other. If one beam were fully loaded (full live load and dead load), and the other carried dead load only, there would be a tendency toward bending, caused by the larger reaction. The same condition would exist when a heavily loaded beam framed in on one flange, and a lightly loaded beam framed in on the other, except that in this case the eccentricity would always be present. When a beam frames in on one side only, similar circumstances would exist. Ordinary cases involving these conditions are not usually investigated in everyday work. There have been cases, however, when such investigations, although neglected, should have been made. This is particularly true when the reactions involved are large. When to investigate eccentricities of this kind becomes a matter of engineering judgment.

There are a number of reasons why ordinary cases of beams framing into columns are not examined for the effects of small eccentricities. For columns in an intermediate story, the load delivered to a particular column by the column above, is assumed to be concentrically applied. This is reasonable, because the column above is presumed to have the necessary strength, if properly designed, to distribute its load uniformly over its cross-section by the time it reaches the one below. The central load is usually large compared with the eccentric load. This has a steadying influence upon the column section as a whole. Another reason is that many design specifications allow an increased value for the combined stress due to direct compression and bending, as discussed later. This allowance will then protect against a great many cases of small eccentricity. The weight of the column itself and its fire-protection materials are naturally assumed to be concentric.

When a load is considerably eccentric, such as may occur in spandrel beam framing, beams carried by brackets, and so on, the columns involved should be carefully investigated for the combined stresses. Such design requires even more "cut and try" methods than the design of a column concentrically loaded, because the additional element of the eccentric moment must be considered. The column must resist the sum of the direct and eccentric loads, as well as the bending induced by the eccentric load. Obviously then, a column section which cannot sustain the sum of all the loads (as if concentric) must be increased, and the problem is to determine how much more metal is required for the moment stresses. This requires assuming a section in excess of that necessary for direct compression.

In Fig. 379, let P = the direct load, R = the eccentric load, and e = the eccentricity, or the

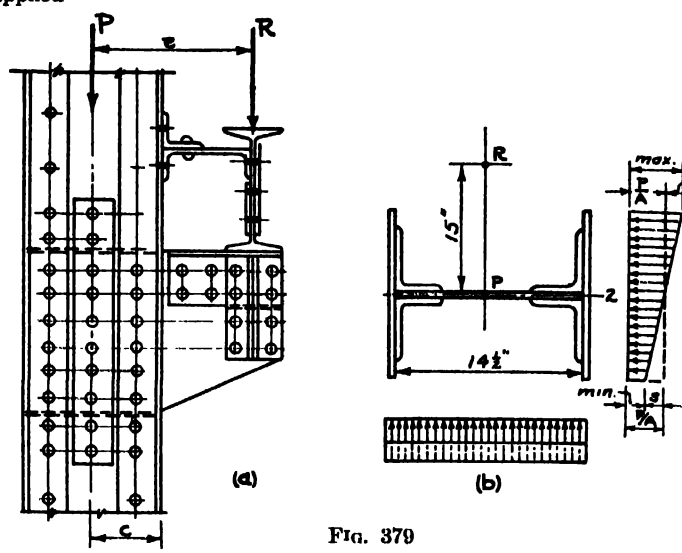


FIG. 379

distance of the point of application of R from the axis of the column. The direct stress is $\frac{P+R}{A}$, in which A = the cross-sectional area of the column. The bending moment, M, is $R \cdot e$. The stress due to this moment may be determined from the usual flexure formula, $M = \frac{s \cdot I}{c}$, or $s = \frac{M \cdot c}{I}$. The value of s is compression on the side toward the eccentric load, and tension on the opposite face. In the usual case, the value of the indirect stress is much less than the direct compression, so that tension seldom becomes an actual condition of stress. The designer is usually interested in obtaining the maximum compression. The moment of inertia, I, in the above formula must of course be referred to an axis perpendicular to the direction of bending. The distance to the extreme fiber, c, must be in the same direction, and naturally to the most remote fiber in compression. The maximum compression is then

$$p\ (\text{max.}) = p\ (\text{direct}) + p\ (\text{indirect}),\ \text{or}$$

$$= \frac{P+R}{A} + \frac{M \cdot c}{I}$$

$$= \frac{P+R}{A} + \frac{R \cdot e \cdot c}{I}. \qquad (S\text{–}63)$$

* If the beam were framed to the web with beam connection angles, the load would be practically on the column axis. If the beam rested on a seat angle instead, the theoretical point of application of the reaction would be at the center of the bearing on the seat angle. This would mean that the reaction would be 2" to 2¼" away from the axis of the column. In practice, such eccentricities are not considered, as the resulting, additional stresses are not serious, particularly in view of the protecting factor of safety.

The tensile stress induced by the eccentric load opposes the direct compression, so that

$$p\ (\text{min.}) = \frac{P+R}{A} - \frac{R \cdot e \cdot c}{I}. \qquad (S\text{–}64)$$

Illustrative Prob. 243a. Design a plate and angle column for the following conditions:

P (direct load) = 210,000#. R (eccentric load) = 40,000#. Eccentricity (e) = 15″. L = 20′-0″. Maximum allowable compression, $p = 19{,}000 - 100\frac{l}{r}$. Refer to Fig. 379 (*b*).

Assume section of:

$$\left.\begin{array}{l} 14 \times \frac{3}{8} \text{ Web Pl.} \\ 4 \text{ L}^s\ 6 \times 4 \times \frac{7}{16} \\ 2 - 14 \times \frac{3}{8} \text{ Cov. Pls.} \end{array}\right\} \begin{array}{l} A = 32.47\square'' \\ I_{1-1} = 1{,}351''^4,\ r_{1-1} = 6.45'' \\ I_{2-2} = 311''^4\ r_{2-2} = 3.09'' \end{array}$$

Total load = 210,000 + 40,000 = 250,000#

$$p\ (\text{direct}) = \frac{P+R}{A} = \frac{250{,}000}{32.47} = 7{,}700\#/\square''$$

$$p\ (\text{indirect}) = \frac{R \cdot e \cdot c}{I} = \frac{40{,}000 \times 15 \times 7.62}{1351} = 3{,}380$$

$$p\ (\text{maximum}) = 11{,}080\#/\square''$$

It will be noted in the above that the values of c and I are about the 1-1 axis, namely that perpendicular to the direction of the bending. The allowable stress is governed by the least radius of gyration, — in this example, $r_{2-2} = 3.09''$.

$$p = 19{,}000 - 100\frac{l}{r} = 19{,}000 - \frac{100 \times 20 \times 12}{3.09}$$

$p = 11{,}300\#/\square''$ allowable

$p = 11{,}080\#/\square''$ actual (from above) O.K.

Use section assumed.

If the maximum allowable and actual stresses had not compared favorably, the column cross-section would be altered until satisfactory results were obtained.

A number of design specifications **allow the compressive working stress,** obtained from the usual column formula, to be **increased 25 per cent.** The reason for this is because only the extreme fibres on the compression side of the column receive the actual maximum compression. Fibres inward from this theoretical plane receive indirect stress only in proportion to their distances from the axis of the column. The compression in the fibres on the side away from the eccentric load is relieved by the amount of tension developed by the bending. Hence, some allowance is reasonable, and the authors recommend the above percentage for use.

Illustrative Prob. 243b. Determine if the maximum compression is safe for the following conditions:

Plate and angle column, $12 \times \frac{1}{2}$ web plate, 4 L^s $6 \times 4 \times \frac{1}{2}$, $2 - 14 \times \frac{1}{2}$ cover plates. L = 16′-0″. P = 410,000#, E = 54,000#. Eccentricity = 9″ from column center on axis 2-2. Maximum allowable stress $= 1.25 \times \left(16{,}000 - 70\frac{l}{r}\right)$.

From tables, $A = 39.00\square''$, $I_{1-1} = 1215''^4$, $r_{1-1} = 5.58''$, $I_{2-2} = 394''^4$, and $r_{2-2} = 3.18''^4$.

$$p\ (\text{direct}) = \frac{464{,}000}{39.00} = 11{,}900$$

$$p\ (\text{indirect}) = \frac{54{,}000 \times 9 \times 6.75}{1215} = 2{,}700$$

$$p\ (\text{maximum}) = 14{,}600\#/\square''$$

$$p\ (\text{usual}) = 16{,}000 - 70\frac{l}{r}$$

$$= 16{,}000 - \frac{70 \times 16 \times 12}{3.18}$$

$$= 12{,}770\#/\square''$$

$$p\ (\text{maximum allowable}) = 1.25 \times 12{,}770 = 15{,}950\#/\square''$$

Column section O.K.

In checking up a given column section for conditions similar to the above problem, the formulas may be varied and a more direct solution made possible. Referring to the formula for the maximum compressive stress,

$$p\ (\text{max.}) = \frac{P+R}{A} + \frac{M \cdot c}{I},$$

multiply both sides of the equation by A, or

$$p \cdot A = (P+R) + \frac{M \cdot c \cdot A}{I}.$$

But $I = A \cdot r^2$. Substituting,

$$p \cdot A = (P+R) + \frac{M \cdot c \cdot A}{A \cdot r^2}, \text{ or}$$

$$p \cdot A = (P+R) + M\left(\frac{c}{r^2}\right). \qquad (S\text{–}65)$$

In the above, $p \cdot A$ = the safe column load (as if applied concentrically). The value, $c \div r^2$ is a constant for any given cross-section. Hence the following rules may be used in conjunction with safe load tables:

(1) Assume column cross-section,
(2) Calculate eccentric moment ($M = R \cdot e$),
(3) Calculate $c \div r^2$,
(4) Multiply M by the value in (3),
(5) Add P, R, and the value in (4), — a result in #,
(6) Refer to safe load tables for given column section and length of column, and compare safe load tabulated with the result in (5).

This might be called an "equivalent central-load" method.

Illustrative Prob. 243c. Check the results in Illustrative Prob. 243*b* by the equivalent central-load method.

(1) Section as given in Prob. 243*b*.
(2) $M = 54{,}000 \times 9 = 486{,}000''\#$.
(3) $c = 6.75''$, $r = 5.58''$, $r^2 = 31.2''^2$, $c \div r^2 = 0.216$.
(4) $486{,}000 \times 0.216 = 105{,}000\#$.

(5) 410,000 + 54,000 + 105,000 = 569,000#.

(6) From safe load tables for given section and $L = 16'\text{-}0''$, $P = 506{,}000\#$. According to specifications given in Prob. 243*b*, this may be increased 25%, or $1.25 \times 506{,}000 = 632{,}000\#$.

Since (6) exceeds (5), column section is safe.

If loads were eccentric on opposite faces of a column, the two bending moments oppose each other in direction, and tend to balance. Hence, only the net bending moment (difference of the two) need be considered in a case of this kind. If two loads are eccentric and are on adjacent faces of a column, the bending moment in each corresponding direction must be considered. The maximum compression will occur at the corner of the column cross-section between the two loads. Its value will be the direct compression plus the compression developed by each of the two eccentric loads, in such a case.

Prob. 243d. Determine a column section for the following conditions:

Plate and angle column with cover plates. $L = 22'\text{-}0''$. Direct load = 250,000#. Eccentric load = 60,000#, applied 12″ from center of column on axis 2-2. Use $19{,}000 - 100\,\frac{l}{r}$ with no increased allowable stress. Hint: try $14 \times \frac{3}{8}$ web plate with 6×4 ∟s and 14″ cover plates.

Prob. 243e. Design a column of the type 22, Fig. 378, for the following conditions:

$L = 15'\text{-}0''$. Direct load = 232,000#. Eccentric load = 43,000#, applied $7\frac{1}{2}''$ from center of column on *y-y* axis (see Fig. 379). Use $16{,}000 - 70\,\frac{l}{r}$ with 25% increased allowable, above the column formula values. Hint: try 10″ channels and 12″ cover plates.

Prob. 243f. Check the design in Prob. 243*d* by the equivalent central-load method.

244. Plate and Angle Columns.

The most common and the most frequently used type of "built-up" structural steel column is one composed of a web plate and four flange angles, as shown in Fig. 372 (*h*), or one of a similar section in which cover plates are added, as in Fig. 372 (*i*). Such a section is sometimes nominally referred to by the width of the web plate, as an "8″ column," "12″ column," etc. This does not describe the section but is merely a convenient name. The 8″ size is the smallest which it is practicable to use for ordinary cases,* and when large web connections are necessary, a 12″ web is usually required, in order to obtain proper details.† The absolute minimum thickness of web plate is $\frac{1}{4}''$, although $\frac{5}{16}''$ is more commonly used as a minimum. For work exposed to climatic conditions, $\frac{3}{8}''$ should be the minimum thickness of all metal. For lower story columns and particularly for heavy loads, web plates $\frac{1}{2}''$ thick or more should be used.

The distance back to back of flange angles is made $\frac{1}{2}''$ greater than the width of the web plate. This allows for any unevenness in the web plate, particularly if the latter is a sheared plate, and presents a more finished appearance. Unequal-legged angles are generally employed for this type of column, with the longer legs outstanding. This arrangement provides the most metal at a maximum distance from the center of the column, for a given amount of material. Hence with the shorter legs of the angles against the web plate, a larger radius of gyration is obtained, and a greater width of column face results. It is desirable to have the width of the legs adjacent to the web plate a 3″ minimum in order to provide ease in riveting. The outstanding legs are made either 5″ or 6″ in the larger columns, so that two gauge lines are available. In the usual section, the least radius of gyration is about the axis coincident with the web. This fact is one which is wise to remember, as it aids in avoiding mistakes. In making up a column section, an attempt should always be made to obtain a "balanced" section, that is, one in which the thickness of the metal in the angles and plates is about the same. Thus a section of a $14'' \times \frac{1}{4}''$ web plate, $4 - 6 \times 4 \times \frac{7}{8}$ angles, and $2 - 14 \times \frac{5}{16}$ cover plates would not be a "balanced" section. Many detail design specifications give certain minimum requirements relative to thicknesses of the component parts. The most common and important ones are:

(1) The thickness of web plate shall not be less than $\frac{1}{30}$ of the distance between the lines of rivets which connect the plate to the angles.

(2) The thickness of the cover plates shall not be less than $\frac{1}{40}$ of the distance between the lines of rivets connecting the plates to the angles.

(3) The thickness of angles, when no cover plates are used shall not be less than $\frac{1}{12}$ of the outstanding leg of one angle.

(4) The maximum aggregate thickness of metal through which any rivet is driven shall not exceed 4 diameters of the rivet.

The common column sections, determined by general practice, usually will conform with the above rules. Figure 380 shows a typical shop drawing of a plate and angle column.

Illustrative Prob. 244a. The design of a plate and angle column by the use of the general theory and formulas, involves the same procedure as already discussed in Art. 242. Refer to Illustrative Prob. 242*a*.

* Small columns are sometimes made with 6″ web plates, but difficult, and sometimes awkward, connections result.

† In mill building work, it is sometimes specified that the width of web plate should be such that the ratio, $\frac{l}{r}$, is minimum in the vertical direction but does not exceed 90.

In practice, where a number of columns are to be designed, the section to be used is not always established by methods such as those discussed in Art. 242, but safe load tables* or diagrams are used. The obvious reason for this is the element of time. In using such tables, one should nevertheless be familiar with the theory, and be able to discriminate the special cases from the ordinary. One should also make certain that the table to be used conforms with the design specifications (type of formula, limiting ratios of slenderness, etc.), which govern. Table 76 illustrates a typical case of how safe loads may be tabulated. Such tables may be found in a number of structural steel handbooks, for a selected line of column sections and various lengths.†

In determining column sections, the first selection may not be the most economical. That is, it may not weigh the least. A section with a wider web plate or larger angles, of thinner metal, may weigh less. The selection, will, of course, be influenced by other considerations, such as the outside dimensions and the space required thereby, dimensions of columns which are directly above, conforming with other typical column sizes in the same story, and so on. A section with no cover

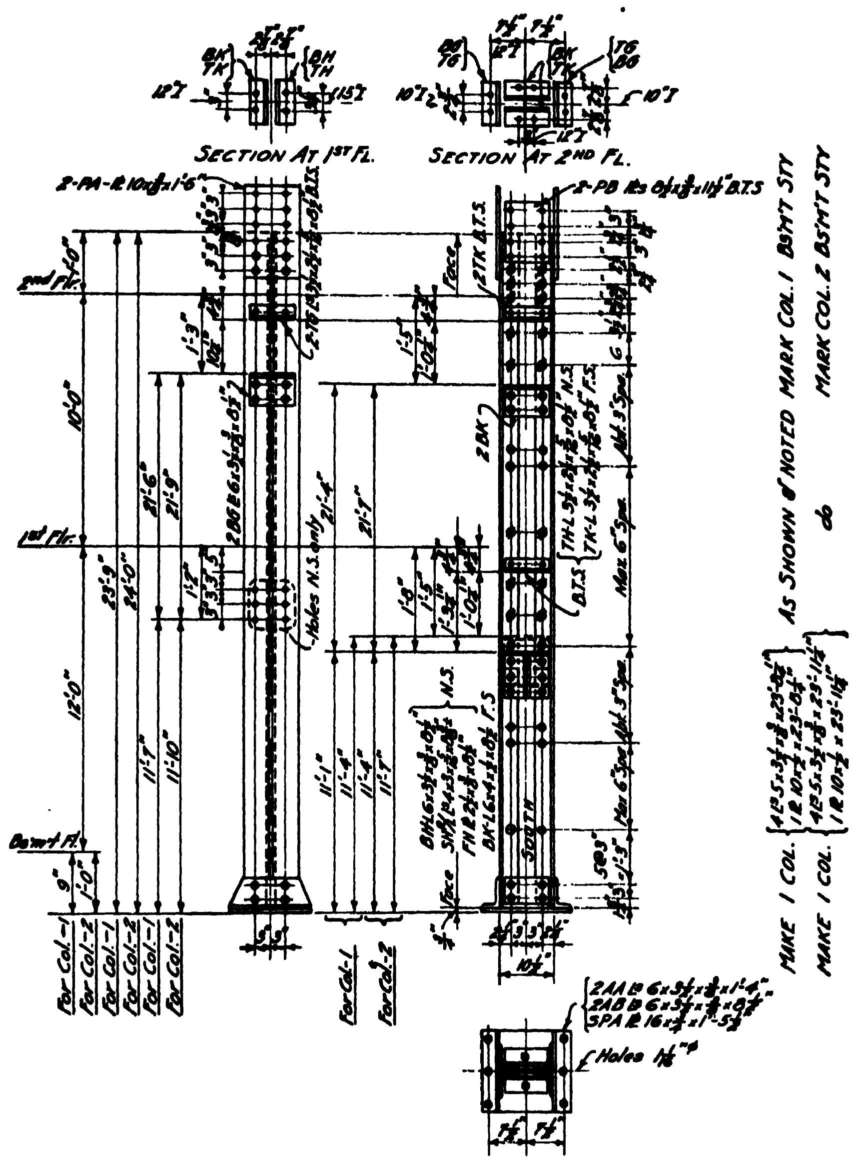


FIG. 380

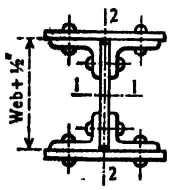
* These tables are constructed by the engineer for the conditions imposed and therefore are merely a tabulation of values involving the above methods.

† The engineer must make sure that any safe-load tables he uses are based upon formulas which agree with the local Building Code.

TABLE 76*

PLATE AND ANGLE COLUMNS

Safe Loads in Thousands of Pounds



Allowable Fiber Stress per square inch, 13,000 pounds for lengths of 60 radii or under; reduced for lengths over 60 radii, $p = 19{,}000 - 100\frac{l}{r}$.

Weights do not include rivet heads or other details.

Effective Length in Feet	Web Plate 12 × 3/8				Web Plate 12 × 1/2				Web Plate 12 × 5/8			
	4 Angles 6 × 4 × 3/8, 2 Plates 14 × 3/8	4 Angles 6 × 4 × 3/8, 2 Plates 14 × 1/2	4 Angles 6 × 4 × 7/16, 2 Plates 14 × 1/2	4 Angles 6 × 4 × 1/2, 2 Plates 14 × 1/2	4 Angles 6 × 4 × 1/2, 2 Plates 14 × 1/2	4 Angles 6 × 4 × 1/2, 2 Plates 14 × 5/8	4 Angles 6 × 4 × 9/16, 2 Plates 14 × 5/8	4 Angles 6 × 4 × 5/8, 2 Plates 14 × 5/8	4 Angles 6 × 4 × 5/8, 2 Plates 14 × 5/8	4 Angles 6 × 4 × 5/8, 2 Plates 14 × 3/4	4 Angles 6 × 4 × 5/8, 2 Plates 14 × 7/8	4 Angles 6 × 4 × 5/8, 2 Plates 14 × 1
11	383	428	458	487	507	553	582	610	630	675	721	766
12	383	428	458	487	507	553	582	610	630	675	721	766
13	383	428	458	487	507	553	582	610	630	675	721	766
14	383	428	458	487	507	553	582	610	630	675	721	766
15	383	428	458	487	507	553	582	610	630	675	721	766
16	379	428	458	487	506	553	582	610	630	675	721	766
17	368	419	447	475	491	542	569	596	613	663	714	763
18	357	407	434	461	476	526	553	579	594	644	694	742
19	346	395	421	447	461	510	536	562	576	625	674	721
20	334	383	407	433	447	495	520	544	558	606	654	700
21	323	370	394	419	432	479	503	527	540	587	634	679
22	312	358	381	405	417	463	487	509	522	568	614	658
23	301	346	368	391	403	448	470	492	504	548	594	637
24	289	334	355	377	388	432	454	475	486	529	574	616
25	278	322	342	363	373	416	437	457	468	510	554	595
26	267	310	329	349	358	401	421	440	450	491	534	574
27	256	297	316	335	344	385	404	422	431	472	514	553
28	244	285	303	321	329	369	388	405	413	453	494	532
29	233	273	290	307	314	354	371	388	395	434	474	511
30	222	261	277	293	299	338	354	370	377	415	454	490
31	211	249	264	279	285	323	338	353	359	396	434	469
32	203	237	250	265	272	307	321	336	341	377	414	448
33	197	228	242	257	264	294	309	323	331	361	394	427
34	191	221	235	250	257	287	301	315	322	351	381	409
35	186	215	229	243	249	279	293	306	313	342	371	399
Area, in.²	20.44	32.94	35.22	37.50	39.00	42.50	44.74	46.94	48.44	51.94	55.44	58.94
I_{1-1}, in.⁴	916	1073	1136	1197	1215	1377	1437	1495	1513	1682	1856	2037
r_{1-1}, in.	5.58	5.71	5.68	5.65	5.58	5.60	5.67	5.64	5.59	5.69	5.79	5.88
I_{2-2}, in.⁴	291	348	368	388	394	451	472	402	499	556	613	671
r_{2-2}, in.	3.14	3.25	3.23	3.22	3.18	3.26	3.25	3.24	3.21	3.27	3.33	3.37
Weight Lbs. per Foot	100.2	112.1	120.1	127.7	132.8	144.7	152.3	159.9	165.0	176.9	188.8	200.7

Safe load values above and to right of upper zigzag line are for ratios of l/r not over 60, those between zigzag lines are for ratios up to 120 l/r, and those below lower zigzag line are for ratios not over 200 l/r.

* From the "Pocket Companion", Carnegie Steel Company.

plates may weigh more than some other one with cover plates, but the extra cost of riveting the cover plates may more than offset the cost of the greater amount of metal. All such factors should be considered, and hence the final choice becomes one of sound architectural and engineering judgment, and a knowledge of steel fabrication. It should be remembered that structural steel columns are usually planned for two-story lengths, so that the load in the lower story is the load governing the design (Art. 259).

Illustrative Prob. 244b. Select an economical cross-section of plate and angle column to carry a load of 400,000# for a 16′-0″ length. Refer to Carnegie Pocket Companion.

Web Plate	Flange Angles	Cover Plate	Weight/Ft.
$12 \times \frac{1}{2}$	$6 \times 4 \times \frac{3}{4}$	....	114.8#
$12 \times \frac{3}{8}$	$6 \times 4 \times \frac{3}{8}$	$14 \times \frac{1}{2}$	112 1

The more reasonable choice is probably the second. Select an economical cross-section for 140,000# and a 12′-0″ length.

Web Plate	Flange Angles	Cover Plate	Weight/Ft.
$8 \times \frac{3}{8}$	$4 \times 3 \times \frac{7}{16}$	. ...	49.4
$10 \times \frac{3}{8}$	$4 \times 3 \times \frac{3}{8}$	. .	46.8
$12 \times \frac{5}{16}$	$4 \times 3 \times \frac{3}{8}$		46.7

Prob. 244c. Design a plate and angle column (use formula $p = 16{,}000 - 70\frac{l}{r}$) to carry a load of 410,000# on a 16′-0″ length. Refer to Fig. 378 for approximate radius of gyration. Assume web plate $14 \times \frac{3}{8}$.

Prob. 244d. Check the safe load given in Table 76 for the following conditions:

Web plate $12 \times \frac{1}{2}$, 4 ⊾s $6 \times 4 \times \frac{9}{16}$, 2 cover plates $14 \times \frac{5}{8}$, length = 21′-0″.

245. "Constant Dimension" Columns.

In the design of structural steel columns for apartment houses, office buildings, hotels, loft buildings, and the like, a comparatively new type has been introduced. These are called "constant dimension columns."* They are so called because the overall dimensions of the cross-sections are constant, within certain limitations for varying loads and areas. Figure 381† illustrates one of a number of series of such columns. Some advantages which may be claimed are:

(1) The extreme dimensions of the column cross-section are known at any point in the height of the stack.

(2) Adjacent columns having different loads may be kept the same size.

(3) The finished dimensions outside the column fireproofing may be maintained constant, thus enhancing architectural details.

(4) The location of column center-lines may be readily and accurately established.

(5) The wall columns may be kept at a constant and minimum distance from the outside faces of the walls. This is an advantage architecturally, as constant pilaster dimensions are possible. Structurally, it is an advantage in framing spandrel beams. (If the overall dimensions of the columns are changed, the columns should be kept flush on the outside.)

(6) Column splices are considerably simplified.

The typical columns may be made up in 10″, 12″, 14″, 16″, 18″, 20″, and special 22″, 24″ and 26″ combinations. Columns should not be less than 10″ for these types, and preferably, 12″ should be used. An 8″ column often produces awkward details. In any case, the section should be a combination of standard shapes and plates that permits fabrication by the usual methods and with the sequence of operations employed in first-class structural shops, and so that adequate and workable details result.

The design of such columns is greatly expedited by the use of safe load tables (see Fig. 381).‡ A column section may be described by its nominal web width and its weight per foot, such as 10 *AB* 117, followed by an index number. If the column schedule gives the make-up of the plates and angles, the index number may be omitted.

In using "constant dimension" columns, as few changes in the outside dimensions as possible should be made. For high buildings in which the lower column sections must necessarily be large, the outside dimensions may be reduced for the upper stories. If changes are made, they should all be at one floor, preferably, or at least, in certain groups. If a 14″ or 16″ column may be used in the lower stories, the outside dimensions should be maintained above, to the top, using smaller sectional areas. Figure 382† gives typical illustrations of column

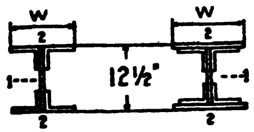
* Originated by the American Bridge Company.

† Excerpted from "Tables Governing Design of Structural Steel in Tier Buildings," edited and copyrighted by the American Bridge Company, New York City.

‡ In "Tables Governing Design of Structural Steel in Tier Buildings," edited and copyrighted by the American Bridge Company, New York City, safe loads are given for selected combinations of sizes for varying lengths. These are given both for the formulas $p = 16{,}000 - 70\frac{l}{r}$ and

$$p = \frac{16{,}250}{1 + \frac{l^2}{11{,}000\, r^2}}$$

12" AB CONSTANT DIMENSION COLUMNS



When specifying these Columns, give either the Section Number and weight per foot, thus: 12 *AB* 121, or the Index Number, thus: INDEX 1215.

Rivets ⅞"

STRESS IN LBS. PER SQ. IN.
16,000 — 70 l/r

To left of heavy line, values of l/r do not exceed 120.
To right of heavy line, values of l/r do not exceed 150.

Section Number	Weight per Foot	Index Number	Width W	Material: Web Plate	Material: Four Angles	Material: Two Cover Plates	Area	Axis-1 Sec. Mod. S	Axis-1 Rad. Gyr. r	Axis-2 Sec. Mod. S	Axis-2 Rad. Gyr. r	Load in Kips, Effective Length in Feet: 8	9	10	11	12	13	14	15	16	18	20	22	24	26	28	Index Number
	34	1201	8½	12×¼	4×3×¼		9.8	39.5	5.01	5.7	1.56	115	109	104	99	93	88	83	78	72	62						1201
	43	1202		5/16	5/16		12.1	48.6	5.01	7.3	1.58	142	136	129	123	116	110	104	97	91	78						1202
	48	1203		5/16	⅜		13.7	56.0	5.06	8.7	1.63	163	156	149	142	134	127	120	113	106	92	78					1203
	49	1204	10½	5/16	5×3½×5/16		14.0	56.9	5.04	11.2	2.03	178	172	166	160	154	149	143	137	131	120	108	97	85			1204
	55	1205		5/16	⅜		16.0	65.9	5.07	13.5	2.08	204	198	191	185	178	172	166	159	153	140	127	114	101	88		1205
	62	1206		5/16	7/16		17.9	74.7	5.11	15.7	2.12	229	222	215	208	201	194	187	180	173	159	145	130	116	102		1206
	66	1207	12½	12×⅜	6×4×⅜		18.9	77.0	5.06	19.2	2.51	252	245	239	233	226	220	214	208	201	188	176	163	151	138	125	1207
	74	1208			7/16		21.2	87.0	5.07	22.6	2.56	283	276	270	262	255	249	242	235	228	214	200	186	172	158	144	1208
	81	1209			½		23.5	96.8	5.08	25.8	2.61	315	308	300	292	285	278	270	262	255	240	225	210	195	179	164	1209
	89	1210			9/16		25.7	106	5.09	29.1	2.64	346	338	330	321	313	304	297	289	280	264	248	231	215	198	182	1210
	96	1211			⅝		27.9	116	5.10	32.2	2.67	376	367	358	350	341	332	323	314	306	288	271	253	236	218	201	1211
	104	1212			11/16		30.1	124	5.08	35.7	2.70	407	397	388	378	369	360	350	341	332	313	294	276	257	238	219	1212
	111	1213			¾		32.3	133	5.08	39.0	2.73	437	427	418	407	398	387	377	367	358	338	318	298	278	258	238	1213
	118	1214			13/16		34.4	141	5.07	42.1	2.75	466	455	445	435	424	413	403	393	382	361	340	319	298	277	256	1214
12 AB	121	1215	14	10×⅜	6×4×½	14×⅜	34.5	145	5.12	48.1	3.12	478	468	459	450	441	431	422	413	403	385	366	348	329	310	292	1215
	127	1216				7/16	36.3	152	5.12	52.1	3.17	504	494	485	475	465	456	446	436	427	408	388	369	350	331	311	1216
	132	1217				½	38.0	160	5.13	56.2	3.22	529	519	509	499	489	479	469	459	450	429	410	390	370	350	331	1217
	138	1218				9/16	39.8	167	5.13	60.4	3.26	555	545	534	524	514	503	493	483	473	452	432	411	391	370	350	1218
	144	1219				⅝	41.5	175	5.13	64.5	3.30	579	569	558	548	537	527	516	505	495	474	453	432	410	389	368	1219
	152	1223	14	10×⅜	6×4×⅜	14×½	43.7	177	5.04	63.2	3.18	607	595	584	572	561	549	538	526	514	492	468	445	422	399	376	1223
	158	1224				9/16	45.4	184	5.03	67.2	3.22	631	620	608	595	584	572	561	549	537	513	489	466	442	419	395	1224
	164	1225				⅝	47.2	191	5.03	71.3	3.25	658	646	633	621	609	597	584	572	560	535	511	487	463	438	415	1225
	170	1226				11/16	48.9	198	5.03	75.3	3.28	682	670	657	645	632	620	607	595	582	557	532	507	482	457	432	1226
	176	1227				¾	50.7	204	5.03	79.4	3.31	708	695	683	670	657	644	631	618	605	580	554	528	502	477	451	1227
	182	1228				13/16	52.4	210	5.02	83.4	3.34	733	720	707	693	680	667	654	641	628	601	575	549	522	496	469	1228
	188	1229				⅞	54.2	216	5.00	87.5	3.36	759	745	732	718	705	691	677	664	650	623	596	569	542	515	488	1229
	194	1230				15/16	55.9	222	4.99	91.6	3.38	783	769	756	742	728	714	700	686	672	645	617	589	561	533	505	1230
	195	1231				1	57.7	228	4.98	95.6	3.40	809	795	780	766	752	738	723	709	695	666	638	610	581	552	524	1231

Fig. 281

selections. In general, the following guides may be used:

Number of Stories	Wind Bracing	Column Sizes	Alternate	
			First 4 Stories	Above
12	Without	10" Full Height		....
16	Without	14" " "		
16	With	16" " "	16"	14"
24	Without	16" " "	16"	14"
24	With	18" " "	18"	16"

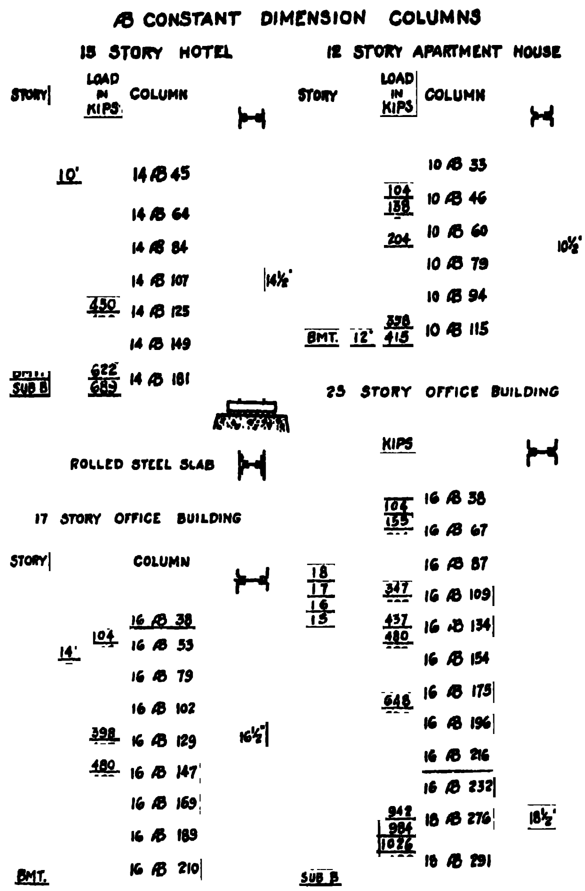


FIG. 382

246. Bethlehem Columns (H Sections).

Probably the most commonly used type of structural steel column is the H section, which is rolled by the Bethlehem Steel Co. Figure 383 illustrates a length of column with its connections ready for shipment.* Figure 384 shows typical shop details of Bethlehem H columns. The main advantage is that only a relatively small amount of fabrication is necessary, — only that for the splices and connections. Other features are the simplicity of details and the fact that all surfaces are accessible, and wide flanges are available for beam connections.† A rolled section is probably stronger than a riveted built-up column having its metal in the shaft weakened by the punching of a large number of holes. Some of the heavier sections of H columns require drilled holes (Art. 23), but gang drills (Fig. 40) may be used to reduce such expense. Special combinations with cover plates, as shown in Fig. 385 (*a*), are now available to avoid local drilling for plates. When the largest available sizes are insufficient, they may be supplemented with extra plates or channels, as in (*b*) and (*c*), although these are not common, particularly (*c*), in which the objection of inaccessibility is again present.

The common sections occur in 8", 10", 12", and 14" sizes of varying weights. Recently, 6" sections have been added. These are advantageous for limited spaces and light loads, although some engineers prefer not to use them, except in special cases, on account of the difficulty of making proper beam connections. Tables of dimensions, weights,

247. Plate and Channel Columns.

A type of structural steel column which is occasionally used is one made up of a pair of channels and a pair of plates, as shown in Fig. 390 (*a*). The ideal section would occur when the values of r_{1-1} and r_{2-2} were equal. The distance

* Courtesy of the New England Structural Co., Everett, Mass.

† A disadvantage of the smaller sizes of these columns is that they have thin webs. This is an argument for the use of conservative stresses in these cases, to prevent web crippling (Fig. 377).

FIG. 383

and structural properties of H column sections for all the variations in size which are rolled may be obtained from handbooks.* Figure 386 illustrates the nature of these tables. The various weights for a given size are obtained by spreading both sets of

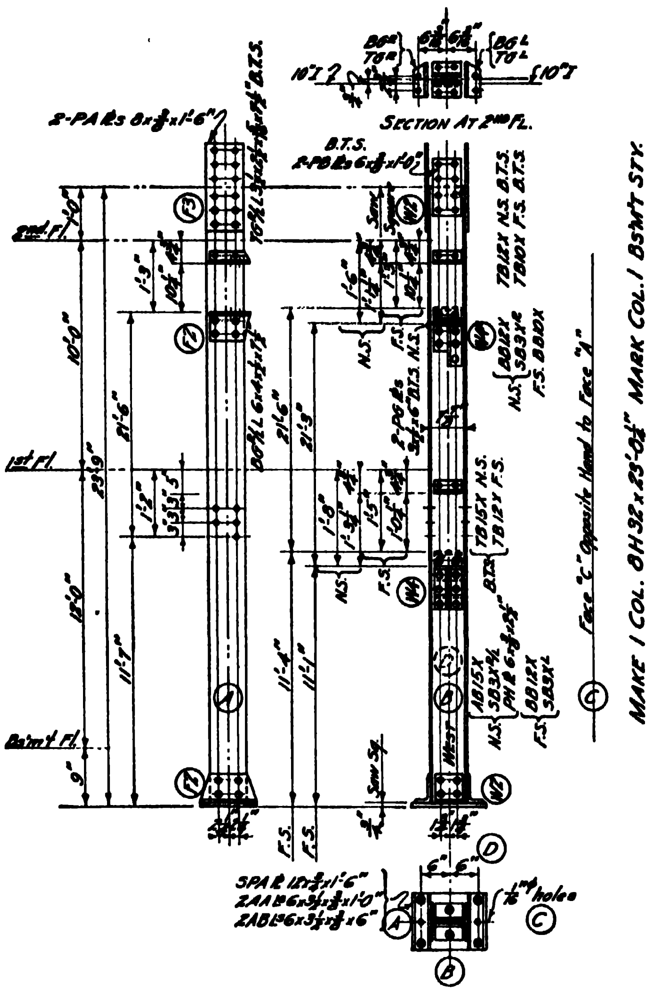


Fig. 384

rolls as shown in Fig. 385 (*d*). The thickness of the web and the width of the flanges is increased equally, and the thickness of the flanges is increased in proportionate amount.

The design of H columns is made in a manner similar to that for other steel columns. In practice, tables of safe loads, or diagrams are frequently used for concentric or symmetrical loading. Figure 387* illustrates the nature of such tables. Figures 388 and 389 give useful diagrams in this respect. It is usually desirable to tabulate the selections for a series of loads on a column stack in order to make a logical selection of sizes, so that the results may be included in a column schedule (Art. 249). The following tabulation* indicates what is meant.

Story	Height of Story Feet	Load on Column, Tons	H Column Section Required					
			Safe Load, Tons	Dimensions, in Inches			Weight of Section, Lbs. per Foot	Section Number
				D	T	B		
16th	12	27	55.0	$7\frac{7}{8}$	$\frac{7}{16}$	8.00	32.0	H8
15th	13	53	81.5	$8\frac{3}{8}$	$\frac{11}{16}$	8.12	48.0	H8
14th	14	79						
13th	13	104	132.2	$10\frac{3}{8}$	$\frac{13}{16}$	10.12	71.0	H10
12th	13	128						
11th	13	151	174.8	$12\frac{1}{4}$	$\frac{7}{8}$	12.08	91.5	H12
10th	13	174						
9th	13	197	219.1	$14\frac{1}{4}$	$\frac{15}{16}$	14.08	114.5	H14
8th	13	219						
7th	13	241	263.8	$14\frac{5}{8}$	$1\frac{1}{8}$	14.19	138.0	H14
6th	13	261						
5th	13	281	310.1	15	$1\frac{5}{16}$	14.31	162.0	H14
4th	13	301						
3d	13	321	341.3	$15\frac{1}{4}$	$1\frac{7}{16}$	14.39	178.5	H14
2d	15	341						
1st	17	363	403.5	$15\frac{3}{4}$	$1\frac{11}{16}$	14.54	211.0	H14
Basem't	12	395						

Some engineers try to confine the selections for big jobs to columns having the same section number. In this way, all the columns can be obtained from the same rolling and quicker delivery is thus probable.

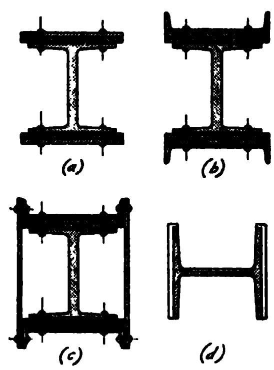


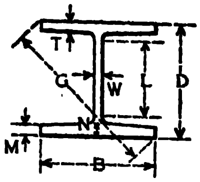
Fig. 385

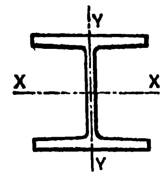
Columns are usually selected for two-story lengths. Where there are no limitations as to size, the largest dimensioned column, having the required capacity, is the most economical.

Prob. 246a. Select an H column to carry a load of 406,000# for a 16′-0″ length.

Prob. 246b. Check the safe column load for an 8 H 34.5 for a 14′-0″ length shown in Fig. 387. Verify your result in Fig. 388.

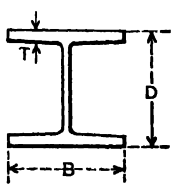
* Handbook of Bethlehem Structural Shapes, Bethlehem Steel Co.





Section Number	Weight of Section, Lbs. per Foot	Dimensions, in Inches								Area of Section, Square Inches	Axis XX			Axis YY		
		D	Nominal T	B	W	M	N	G	L		Moment of Inertia I	Section Modulus S	Radius of Gyration, Inches r	Moment of Inertia I′	Section Modulus S′	Radius of Gyration, Inches r′
	64 5	11¾	⅝	11 92	.39	.567	.683	16¾		19 00	490 0	84.9	5.13	168 6	28 3	2.98
H12	71 5	11⅞	11/16	11 96	.43	.630	.745	16⅞		20.96	556.6	93.7	5.15	188.2	31.5	3 00
	78.0	12	¾	12.00	47	.692	.808	17		22.94	615.6	102 6	5 18	208 1	34 7	3.01

FIG. 386 Dimensions and Properties of Bethlehem Rolled Steel 12″ H Columns



Allowable stress per square inch:
13,000 lbs. for lengths under 55 radii.
$16{,}000 - 55\,\dfrac{l}{r}$ for lengths over 55 radii.

Section Number	Weight of Section, Lbs. per Foot	Dimensions, Inches			Area of Section, Square Inches	Least Radius of Gyration, Inches	Unsupported Length of Columns														
		D	T	B			8 Ft.	9 Ft.	10 Ft.	11 Ft.	12 Ft.	13 Ft.	14 Ft.	15 Ft.	16 Ft.	17 Ft.	18 Ft.	20 Ft.	22 Ft	24 Ft.	26 Ft.
	32 0	7⅞	7/16	8 00	9 17	1.98	59.7	59 7	58 1	56 5	55.0	53.5	52 0	50.4	48 9	47.4	45 9	42 8	39 7	36 7	
HB	34.5	8	½	8 00	10 17	2 01	66 1	66.1	64.7	63 0	61.3	59.7	58.0	56 3	54 6	53 0	51.3	48.0	44 6	41 3	38 0
	39 0	8⅛	9/16	8.04	11.50	2.03	74.8	74.8	73 3	71 4	69 6	67.7	65 8	64 0	62 1	60 2	58 4	54.6	50 9	47.1	43 4

FIG. 387. Safe Loads, in Tons of 2000 Lbs., for Bethlehem Rolled Steel 8″ H Columns. Square Ends

between the channels is made such that nearly equal values of the radii of gyration are obtained, when the dimensions are given to the nearest multiple of ½″. The following combinations are usual:

Size of Two Channels	Width of Plates	Distance Back to Back of Channels
10″	12″	6½″
10″	14″	8½″
12″	14″	8″
12″	16″	10″
15″	16″	9″
15″	18″	11″

In special cases, two web plates may be added, as shown in Fig. 390 (*b*), for very heavy loads. Usually 15″ channels (with 14″ web plates) are the common section for such cases. The weights of a given depth of channel may be varied to give different load capacities, although the minimum weights of a given channel depth are most commonly employed. The thicknesses of the plates may be varied also, to give other load capacities. Safe load tables for various sections and lengths are very helpful in such work.* While the weight efficiency of such columns is good, the inaccessibility for field painting and inspection is a marked disadvantage. Awkward beam connections often result, also.

248. Latticed Columns.

In some of the types of columns shown in Fig. 372, the component parts are held together by a system of lacing (sometimes called latticing). This

* Typical tables are given in the "Pocket Companion" of the Carnegie Steel Company.

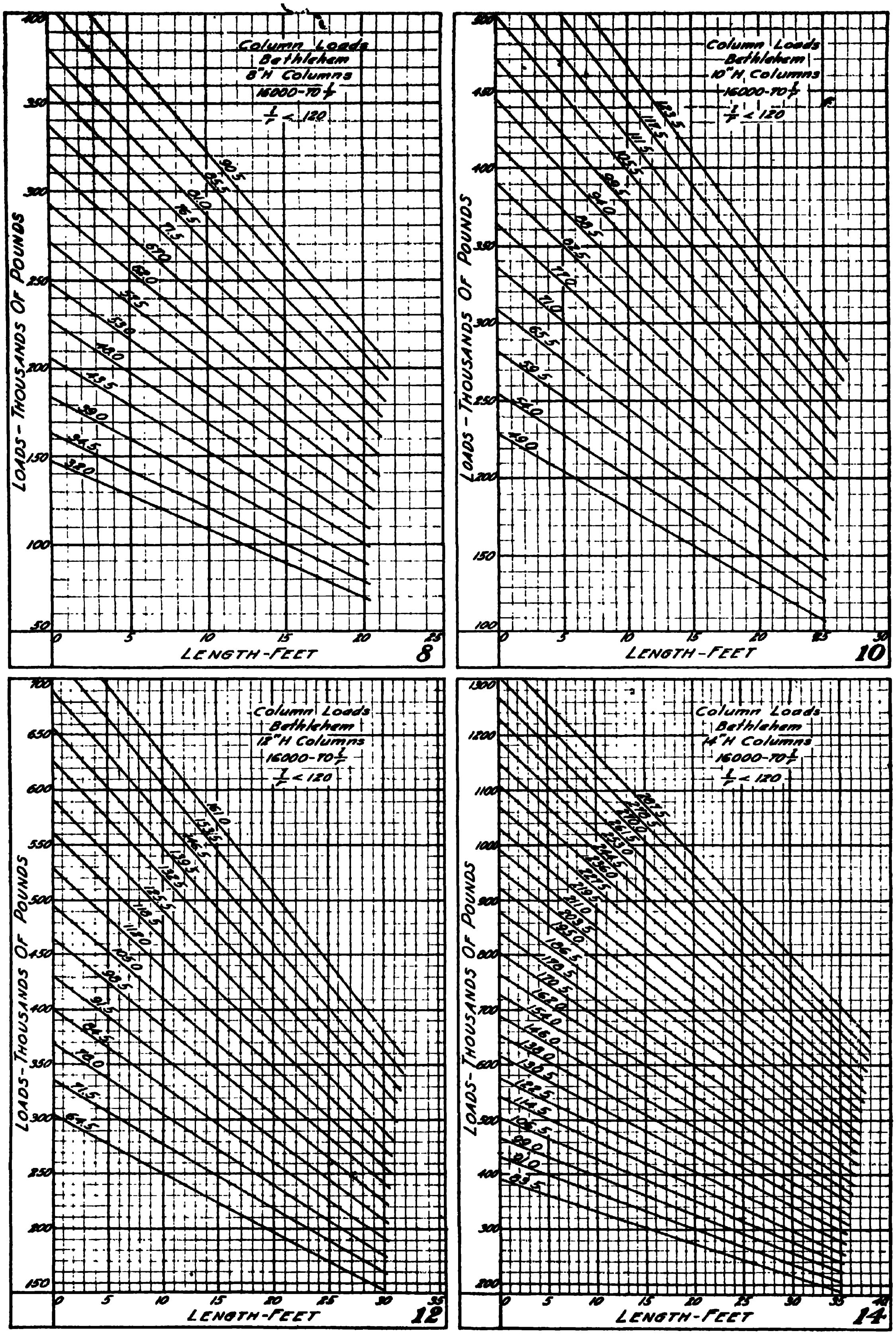

Column Loads
Bethlehem
8"H Columns
16000-70 l/r
l/r < 120
LOADS - THOUSANDS OF POUNDS
LENGTH-FEET
8
Column Loads
Bethlehem
10"H Columns
16000-70 l/r
l/r < 120
LOADS - THOUSANDS OF POUNDS
LENGTH-FEET
10
Column Loads
Bethlehem
12"H Columns
16000-70 l/r
l/r < 120
LOADS-THOUSANDS OF POUNDS
LENGTH-FEET
12
Column Loads
Bethlehem
14"H Columns
16000-70 l/r
l/r < 120
LOADS-THOUSANDS OF POUNDS
LENGTH-FEET
14

FIG. 388

system consists of a series of diagonal bars, either alternating or by crossed diagonals, as shown in Fig. 391 (*a*) and (*b*), riveted to the main parts, and extending across the open sides of the column. The diagonals are supplemented by **tie-plates** near each

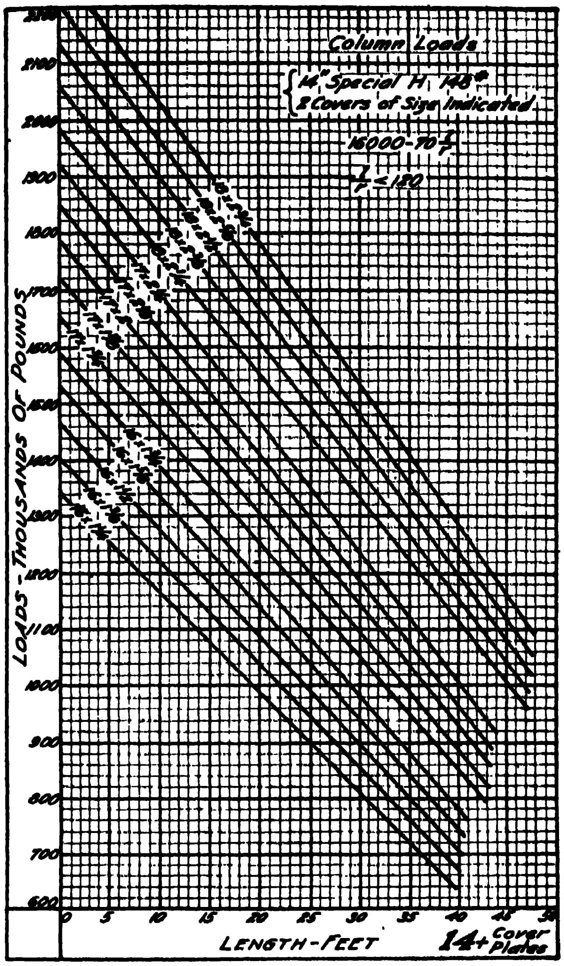
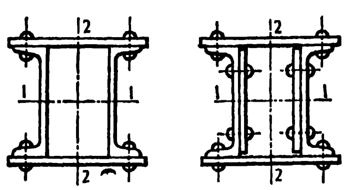


Fig. 389



(*a*) (*b*)

Fig. 390

end and at intermediate points where the lacing is interrupted. Such columns are used principally in mill buildings, bridges and in elevated railroad work, — in the latter case to allow light to pass through.* The design of such columns is similar to that for other steel columns as far as the establishment of the cross-section of the member itself is concerned.† The **distance back to back of angles** or channels usually is made **not less than 12″**. If less, the number of lacing bars increases, as well as

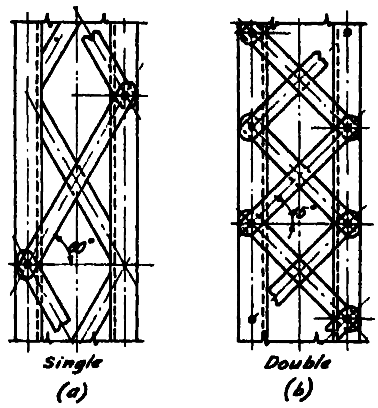


Fig. 391

the amount of riveting, and hence the weight. Narrow widths of the members connected should not be used, as poor connections result. This work is done on the assumption that the component parts

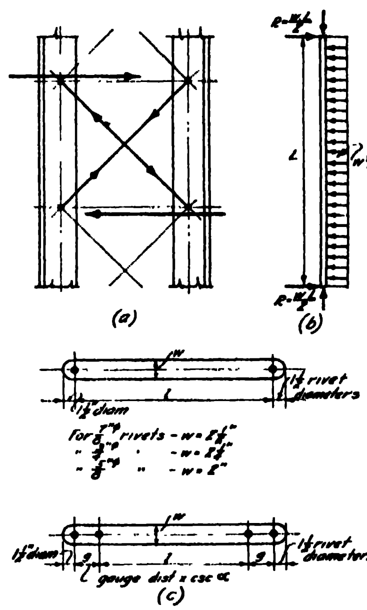


Fig. 392

will be rigidly held together. It is then subsequently necessary for the detail designer to take care of these features.

It is only comparatively recently that the design of lattice bars was considered necessary. Previously,

* Latticed work in general is found more in Europe than in the United States, due to the relative costs of labor as opposed to material.

† Many handbooks give safe loads for latticed columns — see Handbook of the Cambria Steel Co

rules of thumb were used almost entirely. It is good policy to check the requirements of such bracing, as the exceptional case may prove the lacing inadequate if only thumb rules are used.

The purpose of latticing is to provide for the transverse shear caused by the tendency toward lateral flexure in the column.* Diagonal tension and compression are induced by this action, as illustrated in Fig. 392 (*a*). Compression is the governing stress in the determination of the sizes of the lattice bars. The lacing also acts as stiffener bars to prevent bending of the parts during manufacture, shipment and erection, and to resist the shear due to the weight of the member when it is in a horizontal position.

The design of the lacing is based upon the assumption that the column is similar to a beam supported at both ends and uniformly loaded. For such a case, the end reaction and maximum bending as in Fig. 392 (*b*), are respectively

$$R = \frac{w \cdot L}{2} \quad \text{and} \quad M = \frac{w \cdot L^2 \dagger}{8} = \frac{w \cdot L}{2} \cdot \frac{L}{4}.$$

The value of R is the maximum transverse shear and governs the design. Although this shear decreases theoretically, the latticing is made the same the whole length of the column. Substituting R for $w \cdot L \div 2$,

$$M = \frac{R \cdot L}{4} \text{ ft. lbs.} \quad \text{or} \quad \frac{R \cdot l}{4} \text{ in. lbs.}$$

But $M = \dfrac{s \cdot I}{c}$, the resisting moment.

Therefore $$\frac{R \cdot l}{4} = \frac{s \cdot I}{c}.$$

In designing tension members, a stress of 16,000#/□″ is allowed, while in designing compression members, the allowable stress may be determined from

$$p = 16{,}000 - 70\frac{l}{r}.$$

In other words, the design would be similar to that of tension members except for the tendency toward sidewise bending in the compression member. The term, $70\ l \div r$, is the reduction factor, and represents the stress due to bending, in the column formula. Referring again to the equation

$$\frac{R \cdot l}{4} = \frac{s \cdot I}{c},$$

and substituting for s, $70\ l \div r$, and for I, $A \cdot r^2$,

$$\frac{R \cdot l}{4} = \frac{70\ l}{r} \cdot \frac{A \cdot r^2}{c}, \text{ or}$$

$$R = \frac{280\ A \cdot r}{c}, \text{ in which}$$

R = the maximum transverse shear in #,
A = the area of the column cross-section in □″,
r = the radius of gyration of the column cross-section in inches, about an axis perpendicular to the plane of the lacing, and
c = the distance in inches from the center of gravity of the column cross-section to the extreme fibers, in the direction of the lacing.

For symmetrical sections, $c = d \div 2$, and

$$R = \frac{560\ A \cdot r}{d}, \text{ in which}$$

d = the extreme width of the member in inches in the plane of lacing.

Some engineers use the rule of thumb that R (the transverse shear) is 2% of the column load instead.‡

R is only the transverse component of the total stress, S, in the inclined direction, or

$S = R \cdot \operatorname{cosec} \alpha$, in which
α = the angle of inclination of the latticing to a line perpendicular to the longitudinal axis of the column.

The value of α is varied from 60° to 45°, and 45° is **the usual inclination** (especially or double lacing). It should not be made less than 45°. For these values,

$$S = \frac{792\ A \cdot r}{d} \text{ (for 45°), and} \qquad (S\text{–}67)$$

$$S = \frac{650\ A \cdot r}{d} \text{ (for 60°).} \qquad (S\text{–}68)$$

If the lacing occurs in more than one plane (such as a pair of channels laced on each set of flanges), **the stress is assumed to be equally divided among the number of planes of lacing.** The lacing may also be single or double in any one plane. That is, a series of crossed diagonals (double), or alternate diagonals (single), as shown in Fig. 391 (*b*) or (*a*), respectively, may be used. If double lacing is used,

* Where there are eccentric loads on the columns, special designs must be considered for the lacing.

† If a column is subjected to external bending moment in the direction of the plane of the lacing, its value should be added to the indirect moment to obtain the resulting stress.

‡ This varies with different regulations. The A.R.E.A. specification is 2½%. The calculated shear varies with different conditions, and for values of $l \div r$ of 20 to 100, 2% is in excess of the shear, although it makes a good protective rule. From experiments (described in the Proceedings of the A.S.C.E., Vol. LXV, p. 202), the shear varies from 1 to 3% of the load, and when eccentric loads occur, from 2 to 6%.

the stress in a given plane is assumed to be carried equally by each system. Thus the designer must determine how many planes of bracing are to be used and whether single or double lacing is to be employed in each plane. To illustrate, the stress in a lattice bar of a column laced in two planes with double 45° lacing would be $\frac{1}{4}$ S (formula S–67).

The lacing systems are usually provided by **lattice bars.*** These are usually flat bars with rounded ends as shown in Fig. 392 (c), and are made by automatic machines. The minimum width of these bars is governed by maintaining minimum edge distances for the rivet holes. The following tabulation† gives limits in a similar way:

For 15-inch channels, or built-up sections with $3\frac{1}{2}$- and 4-inch angles, $2\frac{1}{2}$ inches ($\frac{7}{8}$-inch rivets).

For 12- 10- and 9-inch channels, or built-up sections with 3-inch angles, $2\frac{1}{4}$ inches ($\frac{3}{4}$-inch rivets).

For 8- and 7-inch channels, or built-up sections with $2\frac{1}{2}$-inch angles, 2 inches ($\frac{5}{8}$-inch rivets).

For 6- and 5-inch channels, or built-up sections with 2-inch angles, $1\frac{3}{4}$ inches ($\frac{1}{2}$-inch rivets).

For flanges $3\frac{1}{2}''$ wide or more, $\frac{7}{8}''\phi$ rivets should be used. One rivet at each end of the lattice bar is sufficient unless the flange connected is more than 5″ wide. A double lattice system should be used if the distance between gauge lines exceeds 15″, unless bars with two rivets at each end are used. The minimum thickness of lattice bars should be $\frac{1}{4}''$ in any case and should not be less than:

$\frac{1}{40}$ the distance between end rivets for single lacing, and

$\frac{1}{60}$ the distance between end rivets for double lacing.

Tie-plates (sometimes called stay-plates or battens) should be used at the ends of the columns and at all intermediate points where the lacing is interrupted. These serve to equalize the distribution of stress. The end tie-plates should have a width in the direction of the column length of not less than the distance between the gauge lines of the rivets. Some detailers use $1\frac{1}{2}$ times this distance. Table 77 gives standard limitations in this respect.

The intermediate tie-plates should not be less than $\frac{1}{2}$ the distance between gauge lines. The thickness of tie-plates should not be less than $\frac{1}{50}$‡ of this distance. Rivets in tie-plates are usually spaced 3″ o.c. As a check upon the latticing, the spacing of the lattice points along the flange, divided by the least radius of gyration of the part connected, should not be greater than the ratio of slenderness of the column as a whole. For extremely large members (greater than 24″ wide), diaphragms (Art. 32) should be used between the webs instead of the lacing, with lengths equal to at least $1\frac{1}{2}$ times the width of the member.

* When excessive diagonal lengths are the case, and large values of the radius of gyration are required, small angles are sometimes used.

† Carnegie Pocket Companion, Carnegie Steel Co.

‡ Some specifications allow $\frac{1}{40}$.

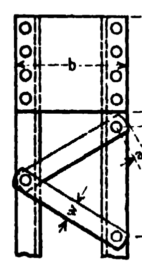
TABLE 77§

SIZE OF LATTICE BARS TO BE USED WITH LATTICED CHANNEL COLUMNS

Depth of Channels	Dimensions of Lattice Bars		Weight of Lattice Bars per Foot	Center of Hole to End (a)	Distance Center to Center of Rivets. (d)	
	Width	Thickness			Maximum	Minimum
Inches	Inches	Inch	Pounds	Inch		
6	$1\frac{1}{2}$	$\frac{1}{4}$	1.28	$1\frac{1}{8}$	$0' - 11\frac{1}{2}''$	$6\frac{5}{8}''$
7	$1\frac{3}{4}$	$\frac{1}{4}$	1.49	$1\frac{1}{8}$	$1' - 1\frac{1}{2}''$	$7\frac{5}{8}''$
8	2	$\frac{5}{16}$	2.12	$1\frac{1}{4}$	$1' - 3''$	$8\frac{11}{16}''$
9	2	$\frac{5}{16}$	2.12	$1\frac{1}{4}$	$1' - 4\frac{1}{2}''$	$9\frac{1}{2}''$
10	2	$\frac{3}{8}$	2.55	$1\frac{1}{4}$	$1' - 6\frac{1}{2}''$	$10\frac{11}{16}''$
12	$2\frac{1}{4}$	$\frac{3}{8}$	2.87	$1\frac{3}{8}$	$1' - 10\frac{1}{2}''$	$13''$
15	$2\frac{1}{2}$	$\frac{3}{8}$	3.19	$1\frac{1}{2}$	$2' - 2\frac{1}{2}''$	$15\frac{5}{16}''$

SIZE OF STAY PLATES TO BE USED WITH LATTICED CHANNEL COLUMNS

Minimum Size of Stay Plates at Ends of Columns			Weight of Minimum Stay Plates	Diameter of Rivets
b	Thickness	l		
Inches	Inch	Inches	Pounds	Inch
$8\frac{1}{4}$	$\frac{1}{4}$	$7\frac{1}{2}$	4.38	$\frac{5}{8}$
$9\frac{1}{4}$	$\frac{1}{4}$	10	6.55	$\frac{5}{8}$
$10\frac{1}{2}$	$\frac{5}{16}$	9	8.37	$\frac{3}{4}$
$11\frac{1}{4}$	$\frac{5}{16}$	12	11.95	$\frac{3}{4}$
$12\frac{1}{4}$	$\frac{3}{8}$	12	15.62	$\frac{3}{4}$
$14\frac{1}{4}$	$\frac{3}{8}$	15	22.73	$\frac{3}{4}$
$16\frac{1}{4}$	$\frac{3}{8}$	15	25.90	$\frac{3}{4}$



Illustrative Prob. 248a. Determine the details for the lacing of a column 30′-0″ long composed of 2-15⊐ 33, $9\frac{1}{2}''$ back to back, with the flanges turned out, as shown in Fig. 393 (a). Use single 60° lattice bars.

For column, $A = 19.8\square''$, $I = 624''^4$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{624}{19.8}} = 5.62''$$

$$d = 9.5 + 2(3.4) = 16.3''.$$

$$R = \frac{560\ A \cdot r}{d} = \frac{560 \times 19.8 \times 5.62}{16.3} = 3820\#$$

§ Excerpted from "Cambria Steel," a handbook published by the Cambria Steel Company, Johnstown, Pa.

There are to be 2 planes of latticing.
R for 1 plane = 3820 ÷ 2 = 1910#. Csc 60° = 1.155.
Stress in 1 bar = 1910 × 1.155 = 2230#.
Flange < $3\frac{1}{2}''$, use $2\frac{1}{4}''$ bar and $\frac{5}{8}''$ ϕ rivets.
Distance between gauge lines = 9.5 + 2 (2) = 13.5″.
Distance between end rivets = 13.5 × 1.155 = 15.6″.

Single lacing O.K., min. $t = \dfrac{15.6}{40} = 0.38''$.

Try $2\frac{1}{4} \times \frac{3}{8}$ bars, $A = 0.84\square''$, $r = 0.11''$.

$$S = A \cdot p = A\left(16{,}000 - 70\frac{l}{r}\right).$$

$$S = 0.84\left(16{,}000 - \frac{70 \times 15.6}{0.11}\right) = 4350\#. \quad \textit{Use } 2\tfrac{1}{4} \times \tfrac{3}{8} \textit{ bars.}$$

$$\frac{l}{r} \text{ for column} = \frac{30 \times 12}{5.62} = 64.$$

$$\frac{l}{r} \text{ for 1 [between lattice points} = \frac{13.5}{0.91} = 15 \qquad \text{O.K.}$$

tie-plates, $t = \dfrac{13.5}{50} = 0.27''$. *Use* $14 \times \frac{5}{16} \times 1'\text{-}4''$.

Intermediate tie-plates. *Use* $8 \times \frac{5}{16} \times 1'\text{-}4''$.

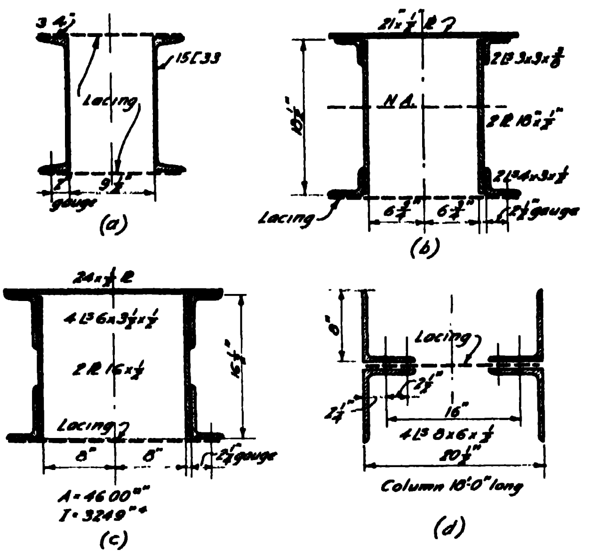


FIG. 393

Illustrative Prob. 248b. Determine the size of lattice bars for the section shown in Fig. 393 (*b*). Use 45° lacing.

For column, $A = 39.2\square''$, $I = 2034''^4$

$$r = \sqrt{\frac{2034}{39.2}} = 7.2''$$

$$d = 2(6.75) + 2(0.50) + 2(4.00) = 22.50''$$

$$R = \frac{560 \times 39.2 \times 7.2}{22.5} = 7000\#$$

Shear resisted in two planes — plate on top and lacing on bottom.

R for one plane = 7000 ÷ 2 = 3500#.
Distance between gauge lines = 19.5″. *Use double lacing.*
R for one bar = 3500 ÷ 2 = 1750#.
Stress in one bar = 1750 ÷ 0.707 = 2480#.
Flange > $3\frac{1}{2}''$, use $2\frac{1}{2}''$ bar and $\frac{7}{8}''$ ϕ rivets.
Distance between end rivets = 19.5 ÷ 0.707 = 28″.

Min. $t = \dfrac{28}{60} = 0.47''$.

Try $2\frac{1}{2} \times \frac{1}{2}$ bars, $A = 1.25\square''$,

$$r = \sqrt{\frac{2.5 \times (0.5)^3}{12} + (2.5 \times 0.5)}$$

$$r = 0.14''$$

$$S = 1.25\left(16{,}000 - \frac{70 \times 28}{0.14}\right) = 2500\#. \quad \textit{Use } 2\tfrac{1}{2} \times \tfrac{1}{2} \textit{ bars.}$$

Prob. 248c. Determine the size of lattice bars for 2-12 [40, 30′-0″ long, 7″ back to back, with flanges turned out (similar to Fig. 393 (*a*)). Use single system, 45° lacing, on each flange. Data: $I = [6.6 + 11.76 \times (0.72 + 3.5)^2] \times 2 = 431.2''^4$, $A = 23.52\square''$, gauge of channels 2″.

Prob. 248d. Determine the size of lacing for the section shown in Fig. 393 (*c*).

Prob. 248e. Determine the size of lattice bars for the column section shown in Fig. 393 (*d*).

249. Column Schedules.

In jobs where a considerable number of columns is involved, the results of the design are usually summarized by a column schedule. Plate 32 gives a typical illustration of such a schedule. The main object is to show the sizes of material to be used in each case. As has been previously stated, **the same sized column is used for two stories** whenever possible, as this method proves to be more economical.* A column schedule generally shows the roof and floor lines with their grades, and the location of the splice lines with respect to them. The relations of the tops of the bases or footings, as the case may be, is also witnessed to the lowest floor grade. **The splice lines are commonly made 2′-0″ above a given floor level.** This allows top clips of the beam connections to clear the lower edges of any splice plates, in the usual case. A typical detail should be worked out, and such a distance verified, for any special instances. The minimum distance from the lowest floor grade to the top of a separate base, or to the top of the footing as the case may be, is made from 1′-6″ to 2′-0″, depending upon the details. This is done so that the base angles or wing plates will not show above the floor if the columns are exposed to view. The distance may have to be a much larger figure than 2′-0″, depending upon the conditions surrounding the footing details.

By the use of diagonal lines in the rectangles formed by the spaces, as shown on Plate 32, the vertical extent of each particular column may be shown. That is, to show what stories it occurs in. Any extensions of some of the columns above the roof level may be indicated, such as for penthouses, towers, and so on. If a column load is transferred to two other columns by means of a girder, this can be clearly indicated by scheduling the three columns

* In some cases, the column erection calls for separate basement columns. This has a two-fold advantage, namely — that it conserves steel because of the usually heavier first floor loads and that it allows early shipment of this portion from stock, the balance coming from the mill at a slightly lower unit price. Some engineers start with the top story column in a single story length because of its usually lighter weight due to light roof loads. They then step the frame down two stories at a time and allow the basement column to become a single length or a unit with the first story column if necessary. The authors advise a comparison of the two methods in any particular case.

involved, side by side, as illustrated on Plate 32. The distance to the top metal of the girder where the column is to rest, may be indicated with reference to the floor line. The dimensions defining the story heights in feet and inches are usually given at one side of the schedule, in addition to the floor grades. The tabulation is of course headed with the column numbers. These numbers appear on the framing plans in small circles near the column conventions.

250. Plan Conventions.

On the plans, a column number indicated thus, ③, shows that the column starts up from that floor, and one shown ④, indicates that the column stops at that floor. If no bars occur with the circles, it signifies that the column is continuous at that plane. The columns on the plans are usually indicated conventionally by H or ⌶, at the intersection of the column center-lines. The two symbols show which way the flanges are to be turned. They are generally used regardless of just what the column sections are to be, whether Bethlehem columns, plate and angle, or otherwise.

There is some variance in practice relative to which direction the flanges of columns are turned. In many cases where rectangular floor plans occur, the girders run in the transverse direction, or parallel to the short side of the building. This allows better light diffusion when the girders are exposed in ceilings. Also they may be made to occur at partitions in many cases, so that only a ledge or shoulder at the ceiling appears. Such framing conforms with economical floor panel design, in which the floor beams run the long dimension of the panel and the girders the short way (Art. 98). Columns which occur at the outside walls are usually turned so that the flanges extend in a direction parallel to the wall. In this way, the columns are supported in their weaker directions by the masonry. If the two above conditions are both established, the girders are then framing between the flanges of the columns for the usual case. Thus the heavier loads are applied so that any bending induced by eccentricity is resisted by the column in its stronger direction. However, it may be argued that if the girders are framed into the webs of the columns, the reactions are more nearly concentric. For large girders and relatively small columns, more expensive details may result for the latter method of framing, because wide flanges of girders may have to be blocked in order to permit them to enter to the column web to receive the connections. Of course, there are exceptions to all rules, and in special cases of corridor framing and so on, it may be advisable to turn columns so that the girders frame into the webs. However, it may be said **in general that it is advisable to frame girders into flanges of columns.**

Columns should always be tied in at least two directions. If untied by the regular framing, tie beams should be introduced. These are usually made 8″ deep minimum, on account of using $\frac{3}{4}''$ rivets connecting the flanges to the seat angles. Otherwise special punching would be required.

In some cases, anchor bolts, when used, are called for in column schedules, although this is not the usual procedure. **Anchor bolt setting plans** are more common. These are practically line diagrams, showing the column center-lines in each direction, the column numbers, and the locations of the bolts at each column. These are commonly indicated by small solid circles and tied to the column center-lines, as indicated in Fig. 394. Typical details are drawn at one side, showing the size and length of the bolts, their imbedment in the footing, projection above the top of the footing, arrangement of washers, and so on.

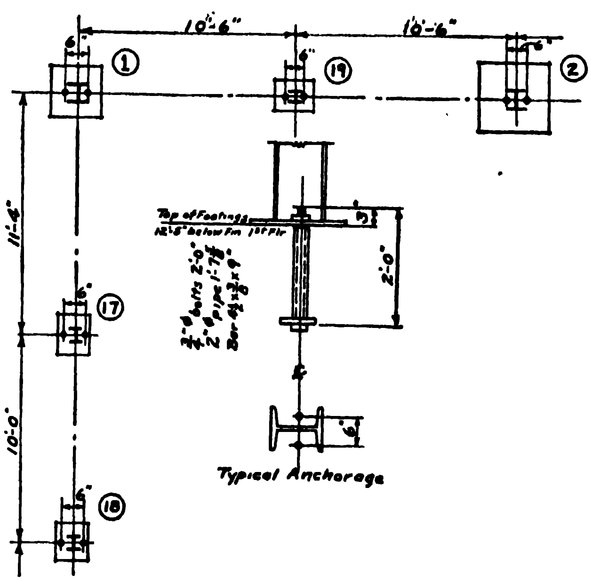


Fig. 394

In special cases, the footings are sometimes included at the foot of a column schedule, showing the plan dimensions, depths, offsets, and steel reinforcement, if any. This is not common, and more often a separate footing schedule is given in conjunction with the foundation plan. The reason for this is because the foundations (including the setting of anchor bolts) are usually a separate sub-contract, apart from the steel work. If not too cumbersome it is advisable to include the anchor bolt setting plan and footing plan with the sheets submitted for an estimate on the steel to avoid any trouble later. Such practice often results in a more satisfactory check-up of the steel support at a very important point.

CHAPTER 22

COLUMN DETAILS

251. General Considerations.

As for all other structural design, the engineer should be familiar with the design of the details, even though he is not ordinarily called upon to make such designs. If the fundamental requirements are understood, the engineer, in the design of the members, is less liable to call for combinations of structural shapes which will result in awkward, impractical and uneconomical details.

Some important considerations in the design of column details are:

(1) The details should not weaken the strength of the column.

(2) They should be simple, to eliminate bending and resulting secondary stresses.

(3) The details should be economical.

(4) A minimum number of rivets, bolts, and connecting pieces, should be used. They should be arranged, insofar as possible, so as to be stressed in shear and bearing only.

(5) All loads should be taken as directly into the column shaft as is practicable.

Erection clearances must be kept in mind. It must be possible to swing the beams to be connected into position. Clearances are required to drive the field rivets. Rivet spacing in general should be as uniform as possible in order to expedite multiple punching. The following discussion follows the good practice covering the above considerations.

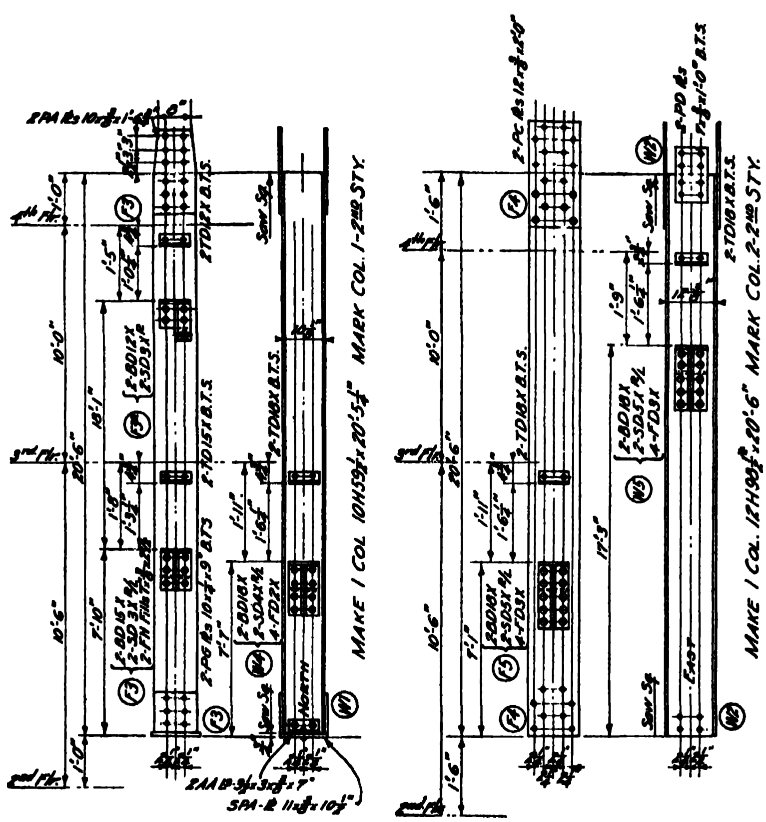


FIG. 395

252. Shop Details.

The actual details of the column lengths and their fastenings are generally made by the fabricating company. Each concern has its own system of shop marking, but all effect the same general purposes. A structural engineer should be familiar with the general features, as he is often called upon to approve the details submitted by the fabricator. Figures 395 and 396 show some typical shop drawings. The details must show the dimensions from the bottom of the columns to the tops of the seat angles where beams are to rest, the relation of the splice to a floor line, relation of top clips to seat angles, the material in the base, gauges, spacing of rivets, stiffeners, and so on. Templates are often used for marking the holes in the webs and flanges of the column shafts. The spacing of rivets must conform to template types. On one view of the web, the direction in which this view is to face should be noted, such as "north," etc. A plan of the column at each floor is generally given. Interference of

rivets in the web and flanges should be studied. In each case each member must bear the same general mark as the structural engineer's drawing, or special erection plans are made to locate each member (Pl. 25.)

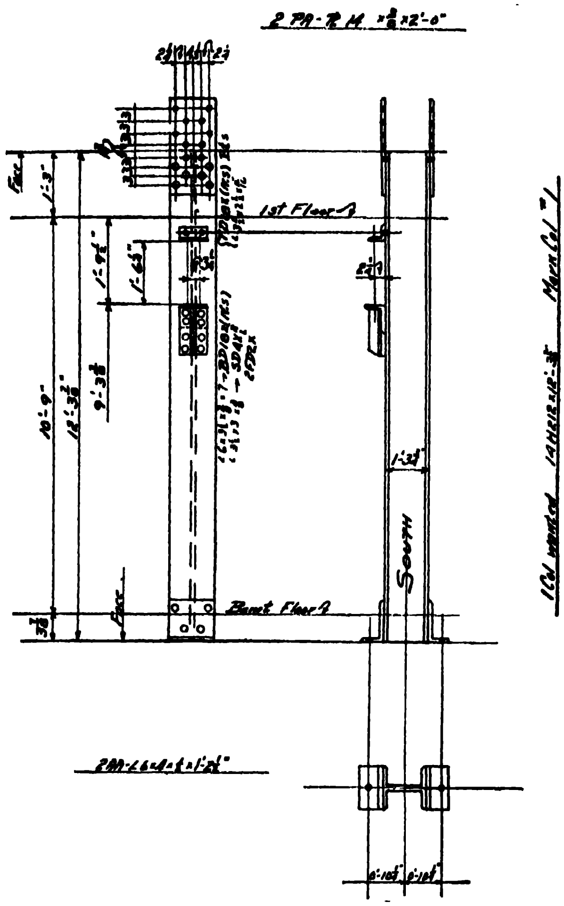


FIG. 396

253. Riveting of Component Parts.

The component parts of a column, particularly when it is built up of structural shapes, must be riveted together in a way which will insure the action of the column as a unit, which is rigid, and which will resist any transverse shears. At points where beams frame in, the local loads must, of course, be developed. In order to insure a uniform distribution of stress over the component parts **at the ends of columns,** a limit is usually specified for the spacing of the rivets.

SPECIFICATION CLAUSE

The pitch of rivets at the ends of built-up compression members shall not exceed four diameters of the rivets for a length equal to one and one-half times the maximum width of the member.

For the usual ¾″ rivets and an average column width of 12″, the above specification is fulfilled by providing **6 spaces @ 3″**. For the remaining portions of the column length (for built-up sections and other shapes with cover plates), the spacing of rivets should be such that there is no tendency toward local buckling of any part, nor any tendency toward a part springing out of line at the edges.

SPECIFICATION CLAUSE

The pitch of rivets shall not exceed 16 times the thickness of the thinnest metal connected, nor 6″ as a maximum. The maximum distance of a rivet from any edge shall not be less than 8 times the thickness of the thinnest metal connected.

In the usual case in practice, the rivets are spaced with a 6″ pitch.

SPECIFICATION CLAUSE

For angles in built-up sections with two gauge lines, with rivets staggered, the maximum pitch in each line shall not exceed 32 times the thickness of the thinnest metal connected. Where two or more plates are in contact, rivets not more than 12″ apart in either direction shall be used to hold the plates together.

No bolted connections are allowed in column details as far as the column itself is concerned (see Art. 23).

SPECIFICATION CLAUSE

In skeleton construction, all splices in columns, all connections of girders or beams to columns, and all connections subject to a reversal of stress shall be made by means of rivets. In all types of construction, splices in girders and chords of trusses and connections carrying heavy stresses shall be riveted. Minor connections such as floor stringers to girders, carrying moderate stresses, may be either riveted or bolted.

254. Seat Angles.

The usual detail to support beams and girders framing into columns is a seat angle, such as at A in Fig. 397 (a). If the beam reaction is small, such an angle by itself may be sufficient to support the load, as at w in Fig. 397 (a). The standard size is 6″ × 4″.* For ordinary cases, the maximum number of rivets which may be driven in the 6″ leg, adjacent to the column face, is 4. If a larger number of rivets is required, then stiffeners are added, as shown in Fig. 397. These are commonly 3½″ × 3½″ in size. The use of stiffeners necessitates fillers, F, to make up the space between the stiffeners and the face of the column (thickness = that of seat angle). Stiffeners may be employed

* Buckling resistances of beams are based upon the bearing on a 4″ outstanding leg of the seat angle, of 3¼″, which is the 4″ less a ¾″ clearance (see Art. 13).

either singly or in pairs, as the various details in Fig. 397 illustrate, according to the number of rivets required. Many detailers prefer to keep the arrangement symmetrical whenever possible. The thickness of these angles will be a function of the rivet allowance and the bending induced in the angle by the beam load.

Top clips, as shown at y in Fig. 397, are used to furnish steadiness to the top flange. These are bolted to the column " to ship " (B.T.S.), and when the beam is ready to be swung into position, they may be removed. When the beam is in place, the top clips may be field riveted. A pair of open holes is left in the top and bottom flanges of the beam. The beam is usually field riveted at these points to provide lateral resistance.* No value in carrying the end reaction is assigned to the rivets in the top clips.

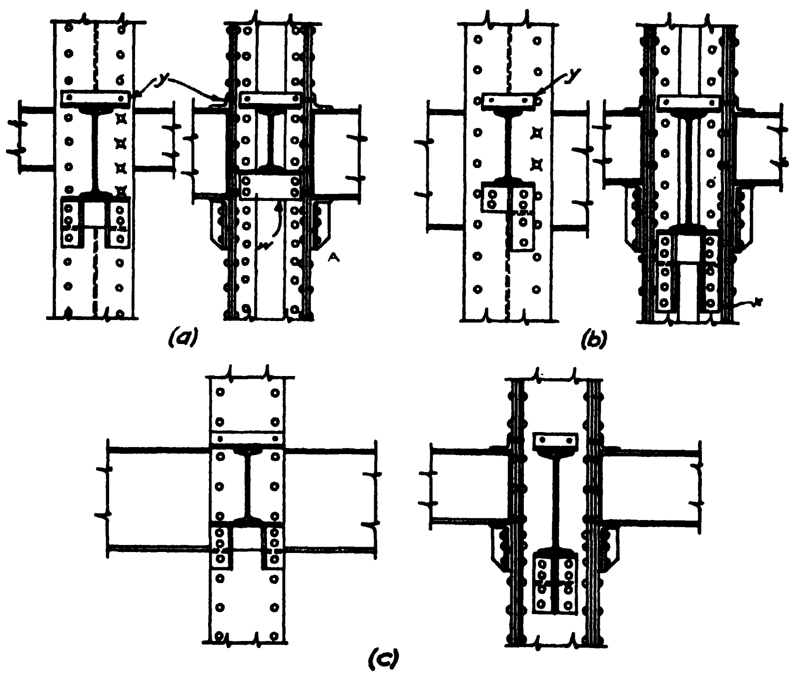


Fig. 397

The required number of rivets is determined by dividing the end reaction of the beam by the controlling value of a rivet (single shear or bearing in the usual case, and generally the former), as discussed in Art. 27.

Illustrative Prob. 254a. How many rivets are required to seat a beam having an end reaction of 18,000#, if the minimum thickness of the column material is ½". Use ¾" rivets, — the common size.

$$\text{Single shear} = 5300\#$$
$$\text{Bearing on } \tfrac{1}{2}'' \text{ metal} = 9000\#$$
$$\frac{18{,}000}{5{,}300} = 3+ \qquad \textit{Use 4 rivets.}$$

If two gauge lines only are available in the column face, as is the usual case, then 4 rivets are the maximum number which may be placed in the 6" leg of the seat angle.

If four gauge lines were available in the column face, 8 rivets might be used, providing that the gauge lines were far enough apart to space the rivets transversely at distances of 3 rivet diameters or more, and providing that the inner rivets did not foul the fillets of the column.

Single shear is the common controlling value of the rivets, so that **the usual maximum end reaction for a seat angle without stiffeners is** $4 \times 5300 = 21{,}200\#$, based upon the resistance of the rivets. However, the seat angle must have sufficient thickness to resist the bending induced. In Fig. 398, if a ½" standard clearance is allowed, the bearing length is 3½". The center of bearing, or the point of application of R, is indeterminate, but the real action is probably similar to that indicated. This may be assumed as at the third-point of the bearing length. The radius of the fillet averages ½". Assuming an average thickness of angle as ½",

$$a = [0.5 + \tfrac{1}{3}(3.5)] - 0.5 - 0.5 = 0.67''.$$

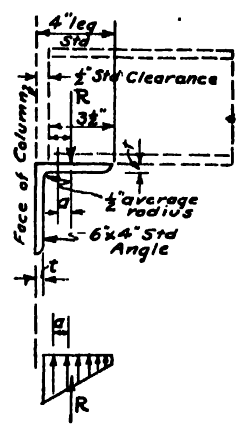


Fig. 398

Illustrative Prob. 254b. If the width of the seat angle in Illustrative Prob. 254a is 8" (across the column face), what thickness of angle is required?

$$M = R \cdot a = 18{,}000 \times 0.67 = 12{,}000''\#$$

$$M_r = \frac{s \cdot b \cdot t^2}{6}. \qquad 12{,}000 = \frac{24{,}000\dagger \times 8 \times t^2}{6}$$

$$t = 0.61'' \qquad \text{Use a } 6 \times 4 \times \tfrac{5}{8}\,\text{L}.$$

* If wind stress connections are to be provided, larger top clip angles and more rivets may be necessary (see Index).

† The value of s may be increased to 24,000#/□" here without being excessive, as the stiffness of the vertical leg and the fillet tend to reduce the effective lever arm.

For average widths of seats, the following approximate values, as far as the resistance of the angles is concerned, may be used as guides:

$\frac{1}{2}''$ seat angles, maximum reaction	=	15,000#
$\frac{5}{8}''$ " " "	=	20,000
$\frac{3}{4}''$ " " "	=	25,000

For reactions larger than 21,000#, stiffeners should be used. In many cases, structural shops work up standard details for such cases. Figure 399 gives a typical illustration. The stiffeners are usually made a nominal size, as the principal function is to accommodate the extra rivets, and to prevent local bending in the seat angle. As far as sectional area in the stiffener angles is concerned, a sufficient amount is usually supplied if the size is made to conform with the rivet spacings* (see Art. 55 for the design of stiffeners). The stiffener angles are usually clipped at a 45° bevel, as shown in Fig. 399, to give a more workmanlike appearance and the upper inside corners are ground off to clear the fillet of the seat angle. For ordinary cases where a pair of angles is used back to back, the outstanding legs are not riveted together. If long angles are required to accommodate the rivets, the outstanding legs may be fastened together with stitch rivets, with 12″ o.c., as a maximum pitch.

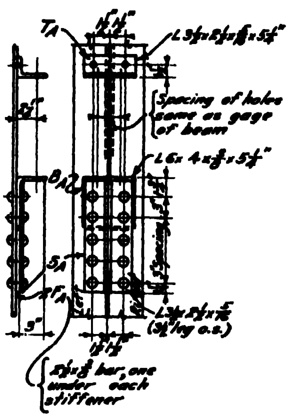


Fig. 399

The bearing area for the beam, supplied on the beam seat, is generally very much in excess of what is theoretically required, as a high unit bearing stress is allowable. It is preferable to have the outstanding leg of a stiffener angle under the web of the beam. The number of rivets in one stiffener angle should be limited, as there is a tendency for the upper rivets to receive the larger portion of the load. Angles should be used for stiffeners whenever possible, although plate stiffeners may be used when clearances are important and dictate their use. An objection to the use of plates is that it is difficult to get good bearing under the seat angle. One objection to the use of seat angle details with stiffeners occurs when the finish lines must be as near the steel frame as possible. This is illustrated in Fig. 400. The case shown in (a) gives an awkward bracket appearance in the finish, but in (b), this is eliminated. Special beam seats may be required for irregular framing, such as those for some spandrel beam connections, as illustrated in

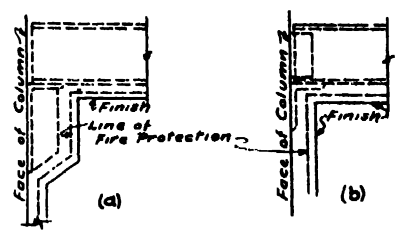


Fig. 400

Fig. 401. The design of the local seat is similar to the above discussion, but the group of rivets at A must be proportioned for the bending due to the eccentric load (Art. 256).

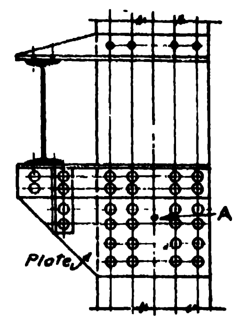


Fig. 401

Prob. 254c. Determine a seat arrangement for a 12 I 31.8, having an end reaction of 19,000#, to frame into the flange of a column composed of a 12 × $\frac{3}{8}$ web plate, 4 angles 5 × 3½ × ½, and 2 – 12 × ½ flange plates. Draw a $\frac{3}{4}''$ scale detail of the arrangement.

Prob. 254d. Provide a seat arrangement for a 24 I 100, having an end reaction of 41,000#, to frame into the web of the column given in Prob. 254c. Draw a $\frac{3}{4}''$ scale detail, labeling the sizes and lengths of all the fittings.

Prob. 254e. Are the details shown in Fig. 402 satisfactory? Make check computations to verify your answer.

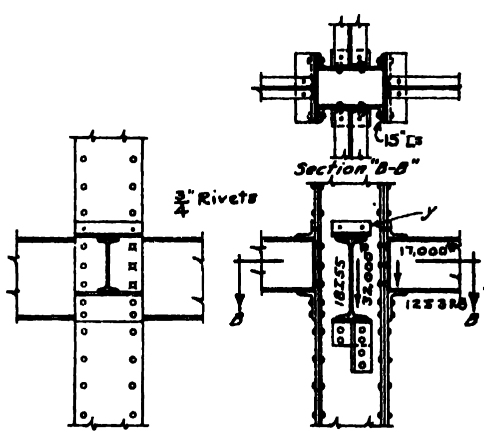


Fig. 402

255. Beam Connection Angles.

In special cases, beams may be framed into the columns with connection angles, as illustrated in Fig. 403. The angles are shop riveted to the beams and then the beams are attached to the column by field rivets. The holes in the outstanding legs of

* The outstanding leg of the stiffener angle is made either ½″ or 1″ less than the outstanding leg of the seat angle.

the connection angles must match the gauge lines in the column shaft. One reason that this type of connection is not commonly used is that the erection is usually quite awkward. It is necessary to swing the beams into position and with the aid of drift pins, insert two or three bolts through the open holes to hold the beams temporarily in place, such as for a case "x" in Fig. 403 (a). On account of this situation, "erection seats" are often used, as shown at

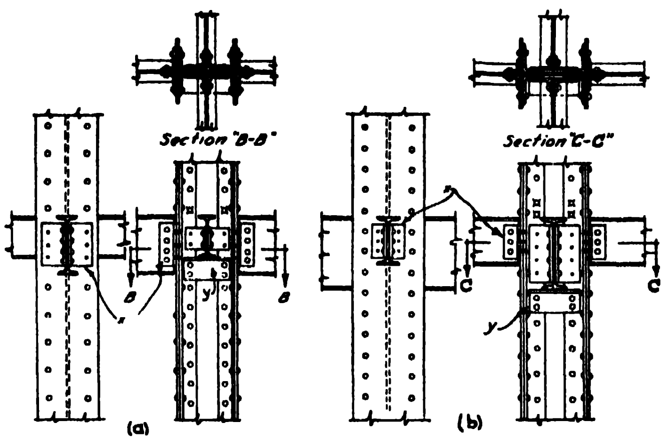


Fig. 403

"y." These are set slightly below the plane where the bottom of the beam is to finally rest (usually $\frac{1}{4}''$). The beam may then be swung into temporary position, shimmed up, and then field riveted into place. In this way, the beam, when finally connected, is free from the erection seat, and hence the latter is not counted upon in a supporting capacity. This is proper, as connections should not be used in which part of the end reaction is calculated as being carried by connection angles and the remainder by a seat angle, if it is at all possible to avoid it. The amount of the reaction carried by each part of the connection in such a case would be indefinite, and if some of the holes did not match properly, the whole reaction might be carried entirely by one part or the other, thus leading to overstress.

The design of the angles serving as beam connections is similar to that discussed in Arts. 28 and 29. Standard connection angles may be used if the holes in the outstanding legs will match the column rivets.

Prob. 255a. If a 12 I 31.8 has an end reaction of 17,000# and frames into the flange of a column, could a standard angle connection be used?

Prob. 255b. If a 15 I 42.9 has an end reaction of 26,000#, can a standard connection be employed to frame into a column? If not, design a pair of angles which are satisfactory.

256. Connections for Eccentric Loads.

In certain instances of framing, beams may not be seated on or framed into, columns directly. Such cases occur in spandrel framing, for crane girder brackets, and so on. The connections which receive such loads are usually subjected to bending. The rivets which support direct loads are proportioned on the assumption that each rivet in the group takes its share of the load (Arts. 72 and 73). This resolves itself into relatively simple design, as the number of rivets required is established by dividing the load by the safe resistance of one rivet.

When a bending moment is exerted upon a group of rivets, however, additional stresses in the rivets are developed. The design of the joint in such a case must be made by "cut and try" methods. A trial number of rivets must be assumed, their arrangement decided, and the maximum stress calculated and compared with the allowable. The number of rivets will certainly be larger than that required for the direct load, but the number to add to the latter is a matter of judgment, and depends upon the magnitude of the moment and the relation of the rivets to each other. A rule of thumb, which may be used as a guide, is to allow one extra rivet for each 50,000″# of bending. This rule is of course not a positive one.

After a trial group of rivets is decided, the next step in the design is to calculate the center of gravity of the group. The direct stress in any rivet (or the reaction to the load) acts in a direction opposite to that of the load. Its value is the load divided by the number of rivets in the group. The bending produced by the eccentric load tends to turn the connection about the center of gravity of the group of rivets as illustrated in Fig. 404. The indirect stress in each rivet is in a direction perpendicular to the radial line drawn to it from the center of gravity of the group. The amount of stress induced by bending in each rivet is proportional to its distance away from the center of gravity of the group of rivets. The stress on the rivet farthest away is hence the largest, and controls the design. This value may be expressed by

$$r = \frac{M \cdot d_n}{I_r}, \text{ in which} \qquad (S\text{–}69)$$

r = the stress on the extreme rivet in #,
M = the bending moment exerted on the group of rivets in in.-lbs.,
d_n = the radial distance from the center of gravity of the group of rivets to the extreme rivet in ins., and
I_r = the "polar moment of inertia" of the group of rivets.

The value of I_r is the summation of the squares of the distances to all of the rivets from their center of

gravity. The value of M is the product of the load and the normal arm from the center of gravity. Formula *S*–69 is derived and more fully discussed in Art. 73.

The resultant maximum stress is then found by combining the maximum indirect stress with the direct stress, each acting in its respective direction. This may be done by using a parallelogram of forces, or using the trigonometric relations.

In Fig. 404 (*a*), there is shown one type of eccentric joint in the form of a gusset plate, which serves as a bracket. The gauge lines are kept standard for the angles whenever possible. The spacing of the rivets is commonly made 3″ or 4″.

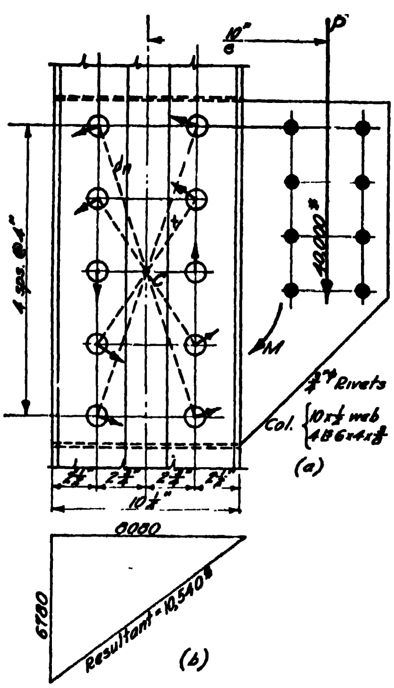


FIG. 404

Illustrative Prob. 256a. Determine whether the arrangement shown in Fig. 404 (*a*) is safe or not, if ¾″ rivets are used.

Double shear = 10,600# per rivet. Enclosed bearing on web = 30,000 × ¾ × ½ = 11,200#. Unenclosed bearing on the angles = 24,000 × ¾ × 2 × ⅜ = 13,500#. The value of 10,600# controls.

$\dfrac{40{,}000}{10{,}600}$ · 3.9, say 4 rivets, required for the direct load.

$M_e = P \cdot e = 40{,}000 \times 10 = 400{,}000''\#.$

$\dfrac{400{,}000}{50{,}000} = 8.$ 4 + 8 12. Assume 10 rivets in the group, spaced as shown (using thumb rule). The center of gravity is at C, since it is a symmetrical group. The polar moment of inertia (I_r) may be calculated by adding the squares of the horizontal distances and the squares of the vertical distances (the square of the hypotenuse of a right triangle is equal to the sum of the squares of the two legs).

Thus $I_r = 10 \times (2\frac{3}{4})^2 + 4 \times (4)^2 + 4 \times (8)^2 = 396.$

For a case where the load is parallel to an axis of the group of rivets, it is simpler to calculate the horizontal and vertical components of the stress.

$d_n = 8''$ (perpendicular to the horizontal component).
$d_n = 2\frac{3}{4}''$ (perpendicular to the vertical component).

$$r \text{ (horizontal)} = \frac{M \cdot d_n}{I_r} = \frac{400{,}000 \times 8}{396} = 8080\#$$

$$r \text{ (vertical)} = \frac{400{,}000 \times 2\frac{3}{4}}{396} = 2780\#$$

$$r \text{ (vertical, due to direct load)} \quad \frac{40{,}000}{10} = 4000\#$$

4000 + 2780 = 6780# total vertical stress.

Combining the horizontal and vertical components graphically, as illustrated in Fig. 404 (*b*), or $\sqrt{(6780)^2 + (8080)^2}$,

r = 10,540# resultant stress on the extreme rivet.
r = 10,600# allowable. O.K.

The actual stress and the allowable stress agree very nearly. If they did not, 8 rivets or 12 rivets could be tried, according to the relation of the stresses.

Illustrative Prob. 256b. A member may frame into another so that the line of the force is inclined to an axis of the group of rivets, as in Fig. 405. Determine whether the joint shown is safe or not.

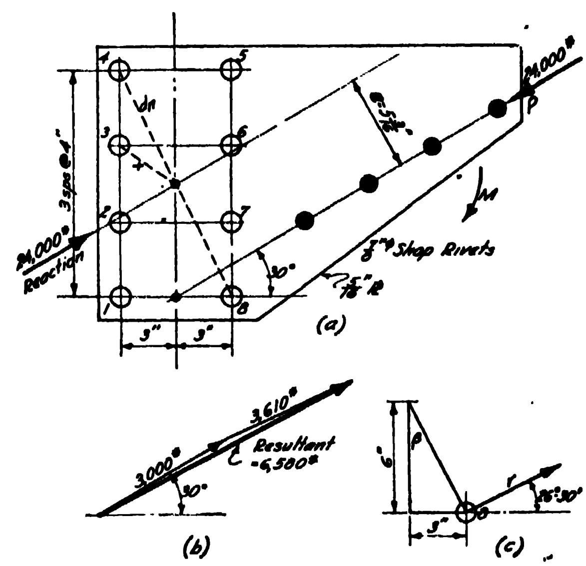


FIG. 405

Single shear = 7220#. Unenclosed bearing on the plate 6560#, which controls.

$\dfrac{24{,}000}{6560}$ = 3+, say 4 rivets for the direct stress.

$M_e = P \cdot e = 24{,}000 \times 5.21 = 125{,}000''\#$

$\dfrac{125{,}000}{50{,}000}$ = 2+. Try 8 rivets for the joint.

$\sqrt{(2)^2 + (3)^2} = 3.6'' = x.$ $\sqrt{(6)^2 + (3)^2} = 6.7'' = d_n$

$4 \times (3.6)^2 + 4 \times (6.7)^2 = 232 = I_r$

$r = \dfrac{M \cdot d_n}{I_r} \quad \dfrac{125{,}000 \times 6.7}{232}$ · 3610# the stress due to the

bending on the extreme rivets 1, 4, 5 and 8. The direct stress $= \frac{24,000}{8} = 3000\#$ on each rivet. Referring to Fig. 405 (c),

$$\beta = \angle \tan^{-1} = \tfrac{3}{6} = \tfrac{1}{2}. \quad \beta = 26° 30'.$$

The stress r (due to bending) acts in a direction perpendicular to the line from the center of gravity of the group to the rivet. The stress r (direct) acts in a direction parallel and opposite to the direction of the 24,000# force. The resultant stress may be obtained graphically as shown in Fig. 405 (*b*).

$$\left.\begin{array}{l} r = 6580\# \text{ actual} \\ r = 6560\# \text{ allowable} \end{array}\right\} 0.3\% \text{ overstressed. O.K.}$$

The excess is under 2% so that the joint may be considered satisfactory. In this particular case, only a small error would be introduced if the direct and indirect stresses had been added as an algebraic sum, because of the small angle involved.

To illustrate further the stresses in the group, Fig. 406 is shown. The indirect stresses on rivets 2, 3, 6 and 7 may be found by direct proportion, using the distances to the rivets instead of the moment formula. Thus

$$r = 3610 \times \frac{3.6}{6.7} = 1940\#.$$

The direct stress is the same for each of the rivets and acts in a direction parallel to the 24,000# force. These are shown by the solid arrows. The indirect stresses are shown by the arrows with the dashed lines. These act at right angles to

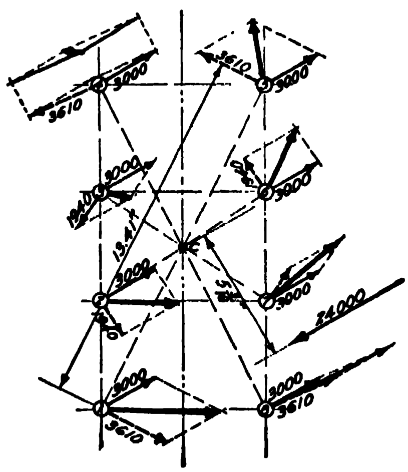


FIG. 406

the lines connecting the rivets with the center of gravity of the group. The resultant stresses are shown by the arrows with the double-lined shanks. It should be noted that if the direct stresses were omitted, the couples in the figure balance, that is, 1940# on rivet 6 is parallel, equal and opposite to the 1940# on rivet 2, the same for rivets 3 and 7, and for the 3610# forces on rivets 4 and 8, and 1 and 5. The direct stresses also form a couple, that is

$$\underset{\text{(reaction)}}{8 \times 3000 = 24,000\#} \qquad \underset{\text{(force)}}{F = 24,000\#}$$

The moments of each of these forces about each other obviously balance. If the moments of the couples formed by the indirect stresses are added, they will balance the external moment. Thus

$$\sqrt{(6)^2 + (12)^2} = 13.41'' \qquad \sqrt{(6)^2 + (4)^2} = 7.24''$$

$$\begin{array}{rl} (3610 \times 13.41) \times 2 = & 96,800 \\ (1940 \times \ 7.24) \times 2 = & 28,200 \\ \hline M_r = & 125,000''\# = M_e. \end{array}$$

The tendencies of these two moments are opposite to each other. Hence complete equilibrium exists, and $\Sigma H = 0$, $\Sigma V = 0$, and $\Sigma M = 0$. Rivet 8 is the only rivet which is stressed to a maximum. Rivet 4 is stressed the least, and the others to values in between the two extremes. It should be obvious then that the center line of stress of a member framing in should pass through the center of gravity of the connection whenever possible.

Illustrative Prob. 256c. Determine whether the bracket shown in Fig. 407 is safe as far as the shop rivets are concerned.

Double shear = 10,600#.

Enclosed bearing on plate = $30,000 \times \frac{3}{8} \times \frac{3}{4} = 8440\#$.

Unenclosed bearing on angles = $24,000 \times 2 \times \frac{3}{8} \times \frac{3}{4} = 13,500\#$. 8440# controls.

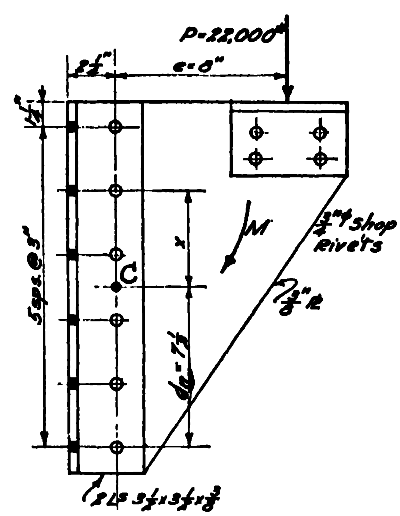


FIG. 407

$$2[(1\tfrac{1}{2})^2 + (4\tfrac{1}{2})^2 + (7\tfrac{1}{2})^2] = 158 = I_r.$$

$$M = 22,000 \times 8 = 176,000''\#$$

$$r_h = \frac{176,000 \times 7.5}{158} = 8350\# \text{ due to moment.}$$

$$r_v = \frac{22,000}{6} = 3670\# \text{ due to shear.}$$

$$r = \sqrt{(8350)^2 + (3670)^2} = 9150\#$$

$$r \text{ (allowable)} \qquad = 8440\# \text{ not safe.}$$

The joint must be revised. A ½" plate would increase the bearing resistance of the shop rivets to 11,250#. The controlling value is then 10,600# and the joint would be satisfactory on this basis. An alternate method would be to use 7 rivets.

At the Load. $P = 22,000\#$.

$$\frac{22,000}{10,600} = 2+. \quad \text{Use 4 rivets in seat angle.}$$

Moment at edge of connection angles,

$$M = 22{,}000 \times 6.75 = 148{,}600''\#$$

$$148{,}600 = \frac{s \times 0.5 \times (17.5)^2}{6} \qquad s = 12{,}700\#/\square''$$

Extreme fiber stress in plate O.K.

$$\text{Shear in plate } \frac{22{,}000}{12{,}000} = 1.8\square'' \text{ net, required.}$$

$$(17\tfrac{1}{2} - 6 \times \tfrac{7}{8})\,\tfrac{1}{2} = 6.13\square'' \text{ actual. O.K.}$$

Prob. 256d. If the load in Fig. 404 (*a*) were 60,000#, arrange a group of rivets to carry the load safely.

Prob. 256e. If the load in Fig. 405 (*a*) were 18,000# and the eccentricity were 8", how many rivets would be required?

Prob. 256f. How many shop rivets would be required in Fig. 407 if P = 39,000# and e = 6"? What provision should be made for the field connections where the open holes are shown?

257. Column Caps.

In special cases, beams may be seated on the tops of columns of one-story length or on top of the upper sections of multi-story columns, as illustrated in Fig. 408. In general, **this is not advisable** and should be avoided whenever possible. It is far better to carry the column up a little further and connect the beams or trusses to the web or flange, as the case may be. If caps are used, the beams or trusses must be properly stayed in a transverse direction. One advantage is that the beam reactions may be brought nearer to the center of the column, but this is more than offset by the lack of stiffness of the horizontal frame.

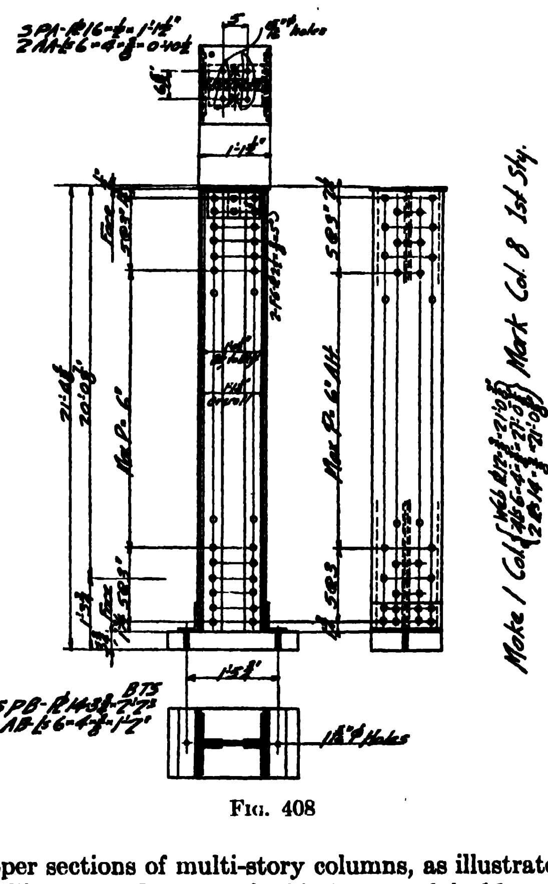


FIG. 408

258. Support of Concrete Girders.

Occasionally in some buildings, concrete-framed floors are used in conjunction with structural steel columns, encased with a concrete fireproofing. This results in the need of the provision for the support of the concrete beams by brackets. The details which are frequently used are shown in Fig. 409. The detail should always provide continuity of the reinforcement, either through or around the column, as shown, and the bearing plates should be fitted with clips or anchor holes for the fastening of the reinforcement, as shown at A. When the reinforcement is passed through the column, the engineer must be careful to provide sufficient area for compression on the net section, or insure adequate protection by encasing the entire column in concrete.

259. Splices.

"Ideal" construction using structural steel columns would occur if one continuous set of sections of the same size could continue the full height of the column stack. This, of course, is not practicable nor economical, and it is necessary to splice the different sections together. The splices should be such that the column, for its full height, will act as a unit as nearly as possible.

It is common practice to have a column length run two stories in height. One-story lengths would offer a saving in the shaft material, but this is usually more than offset by the expense of an extra splice and the additional erection cost. Three-story lengths would be difficult to erect, and the excess material in them would more than balance the saving in an extra splice. Experience has shown the two-story length to be the economical one in general, and it conforms with "carload lengths," as it may be shipped conveniently on flat cars.

The splice lines are commonly placed above the floor line, 2'-0" being the usual dimension, although this may be increased in special cases. This is done so that the details of the splice will not interfere with those of the beam or girder connections, as illustrated in Fig. 410.

The ends of two abutting column sections are accurately milled in practically all cases to insure good bearing. This means that the upper story column load is developed by direct bearing. The purpose of the plates used at a splice is then to hold the sections in line, provide some lateral stability and stiffness, resist any local bending stresses, and serve as a general aid to make sure

that the upper section bears on the lower. All splices must be riveted to insure the best action possible. The outside splice plates are shop riveted to the lower section,* while the filler plates (used to make up the offset between the two columns) are shop riveted to the upper section, with open holes to receive the splice plates when erected. Usually the top row of holes in the lower section are left open for field rivets, as shown at *A* in Fig. 410. This is done so that some adjustment is possible when the upper section is erected upon the lower one.

Column splices may be divided into two general groups: first, those in which the sizes of the two columns change only slightly; and second, those in which there is a considerable change in either size or section. In the first group (such as in Fig. 410), sufficient bearing area is obtained, so that plates are all that are necessary to make up the splice. For the second type of splice, a bearing plate should be used, as shown in Fig. 411 (*c*). This may be made of nominal thickness,† and serves to transfer the load from the upper section over the lower one. The bearing plate is usually fastened by 3×3 clip angles as shown. If the two columns to be spliced are of different cross-sections,‡ stiffeners may be introduced, as shown dotted in Fig. 411 (*a*).

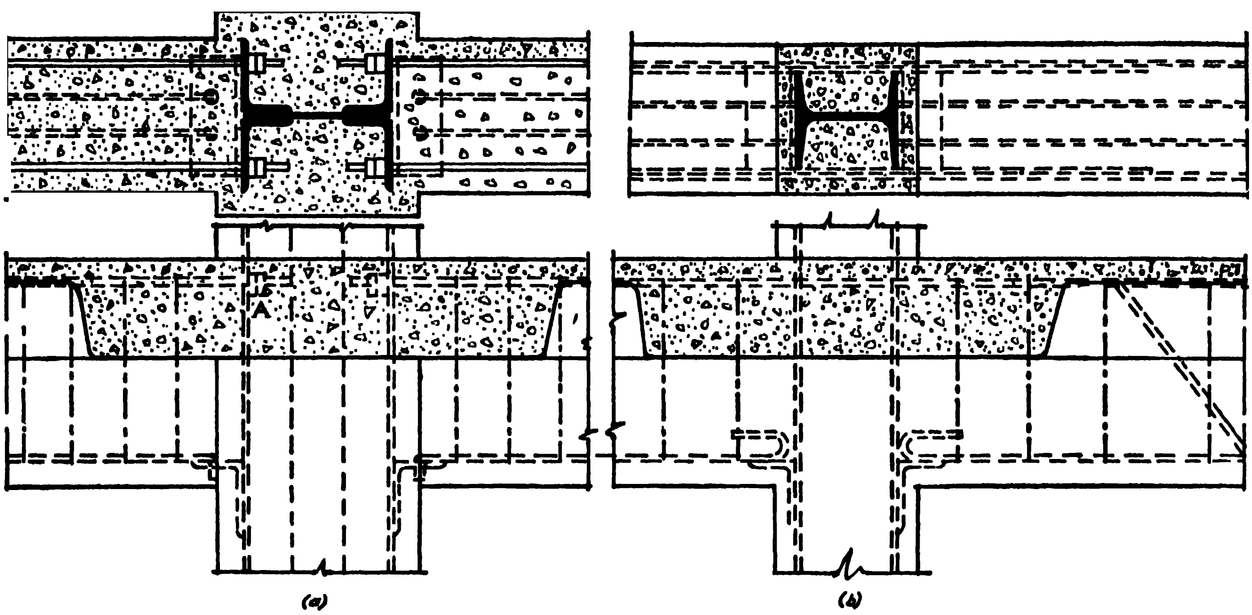


FIG. 409. SUPPORT OF CONCRETE GIRDERS AT STRUCTURAL STEEL COLUMNS

(*a*) anchorage through column flange and at seat angles
(*b*) passing steel outside of column and resting positive steel upon seat angles

* If it so happened that the lower section of column had an extra cover plate, this may be lapped to serve as a splice plate.

† Usually a ½″ or ¾″ plate is sufficient. The thickness may be checked for bending in special cases.

‡ Such a change of sections should be avoided wherever possible.

"Constant dimension" columns (Art. 245) are a distinct advantage in minimizing the cost of splices.

The number of rivets (and consequently the length of the plates) is a matter of judgment. A minimum of two rows each side of the splice line should be used, not counting the rivets which are used to attach the filler plates. The latter should have one row for each intermediate plate and two rows for the one adjacent to the upper column section, as illustrated at *B* and *C*, respectively, in Fig. 411 (*c*). The spacing of the rivets in the direction of the column length is generally made 3″. The following minimum arrangements for the rows of rivets each side of the splice line may be used as a guide:

Type of Column	Number of Rows
Plate and angle — no cover plates	2
Plate and angle — with cover plates	4
Channel	4
Bethlehem — no cover plates	2
Bethlehem — with cover plates	4

As has been stated above, no great lateral resistance is required of the plates and rivets in the average splice. Even if some small eccentricity were developed, or uneven bearing stresses existed because of some eccentric load immediately above, it would be seldom that actual tension would be developed in the steel. In such cases the plates would not be transmitting any great stress.

If a case of an eccentric concentrated load occurred at the splice, such as when the upper section is off center with respect to the lower one, a special investigation should be made. The splice should be designed to resist a moment of:

$$M = \frac{(f - p)\, A \cdot r^2}{c}, \text{ in which} \qquad (S\text{-}70)$$

f = the maximum allowable fiber stress in #/□",
p = the average unit stress in #/□",
M = the bending moment in in.-lbs.,
c = the distance of the extreme fiber from the column axis in ins.,
A = the area of the column cross-section in □", and
r = the radius of gyration of the column cross-section about an axis normal to the direction of bending, in inches.

If the ends of the column sections are not milled (faced) for splicing, the splice material and the connecting rivets must resist the full load. Figure 412 illustrates how a fictitious splice of such a kind would appear. It is obvious that no economy would result, and that milling the ends of the columns would be much cheaper.

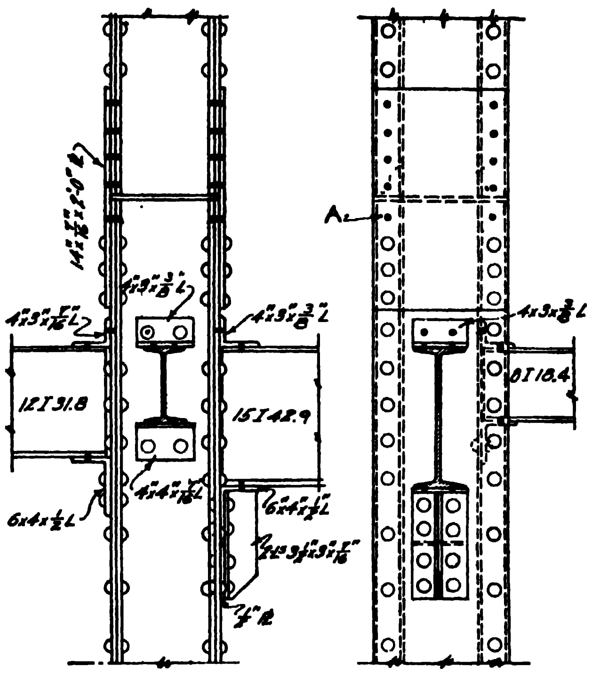


FIG. 410

For small columns carrying moderate loads (usually not greater than 3 or 4 stories high), bases made of **steel plates and angles** (Art. 261), riveted together, may be used, as shown in Fig. 413 (*a*). When the projection of the base plate beyond the edge of the column exceeds 5" or 6", the thickness required for the plate becomes excessive for ordinary stock material, and some other form of base must be used. One of the most common methods is to use **rolled steel slabs** (Art. 262). These are simply thick plates, specially rolled, which can resist more

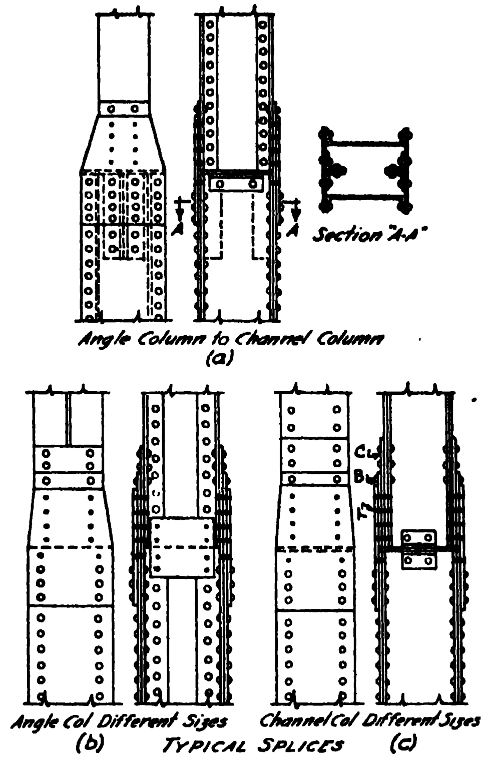


FIG. 411

upward bending. Occasionally, cast-iron plates (Art. 265) have been used.

For heavy loads, **cast-iron pedestals** (Art. 264) are sometimes employed, as shown in Fig. 413 (*b*). Sometimes pedestals are made of cast steel in order to gain the advantage of increased flexural and shearing stresses. Another scheme is to make up a series of rolled steel slabs, as illustrated in Fig. 421.

260. Column Bases in General.*

There are a number of kinds of bases used to distribute bottom-story column loads on to the footings. In each case, the size of the base plate must be such that the allowable pressure on the bed is not exceeded. Footings for steel columns are usually of concrete or of steel grillages. In the first, a safe pressure of 500#/□" is common, and in the second, the base is bedded on a layer of grout from ½" to 1" thick on top of the upper tier of grillage beams, so that a safe pressure of 1000#/□" may be used. There is no exact theory for analyzing bases, but the component parts are checked for flexure and shear, based upon assumptions.

* The ensuing discussion relates only to taking the load on to the footing. The design of the foundations is a subject in itself and requires considerable study apart from the column details. Refer to any standard text on the design of foundations.

261. Riveted Steel Bases.

One form of column base which may be used for columns carrying moderate loads is to employ a steel bearing plate on the bottom, supplemented by shoe angles on the flanges and clip angles on the web. For larger loads, or when the column is to rest upon masonry or concrete, wing plates may have to be used to supply additional stiffness, as shown in

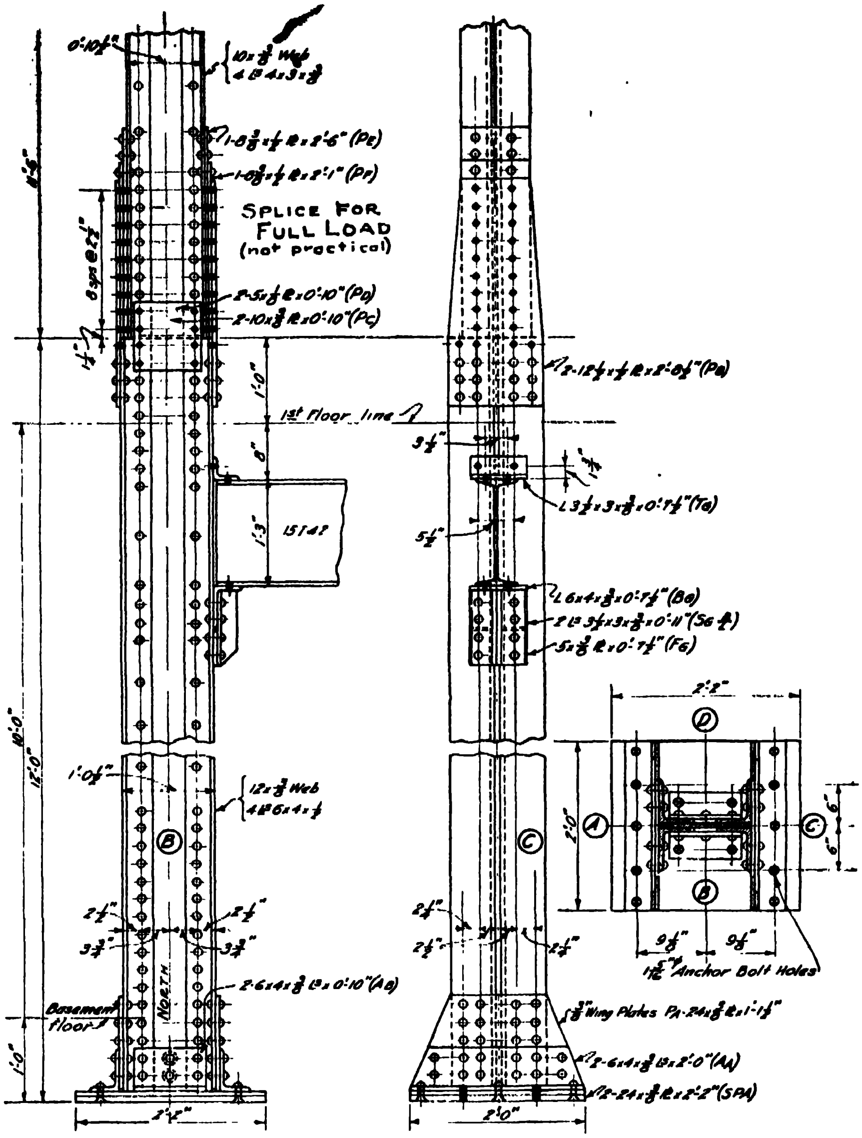
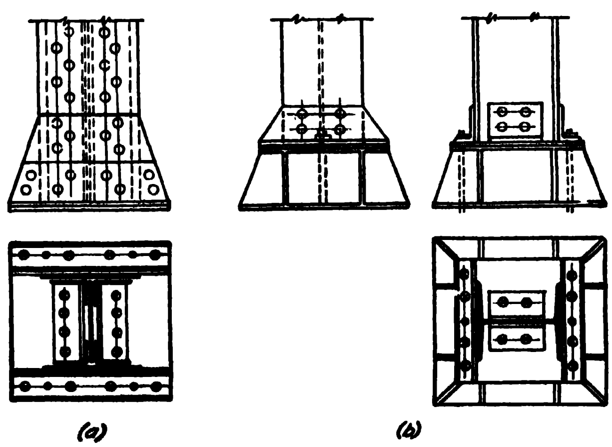


FIG. 412



(a) steel (b) cast-iron

FIG. 413. COLUMN BASES

Fig. 414 (*a*). For columns resting upon rolled steel slabs, only shoe angles are necessary, as illustrated in (*b*) and (*c*).

The design of built-up steel bases involves some judgment, and certain arbitrary methods of arriving at some of the dimensions must be used. The size of the base plate in plan is of course determined by the bearing requirements. The ideal shape is square, although the outside dimensions of the column will be influential. In regard to other sizes, the following tabulation may be of value as a general guide (see Fig. 414 (*a*)):

Light columns:	
Bearing plate	½″ thick
Clip angles	4 × 3 × 5/16
Shoe angles	6 × 3½ × 3/8
Wing plate	5/16″ thick
Medium columns:	
Bearing plate	¾″ thick
Clip angles	4 × 3 × 3/8
Shoe angles	6 × 3½ × 7/16
Wing plate	3/8″ thick
Heavy columns:	
Bearing plate	1″ to 1¼″ thick
Clip angles	6 × 4 × ½
Shoe angles	6 × 4 × ¾
Wing plate	7/16″ or ½″ thick

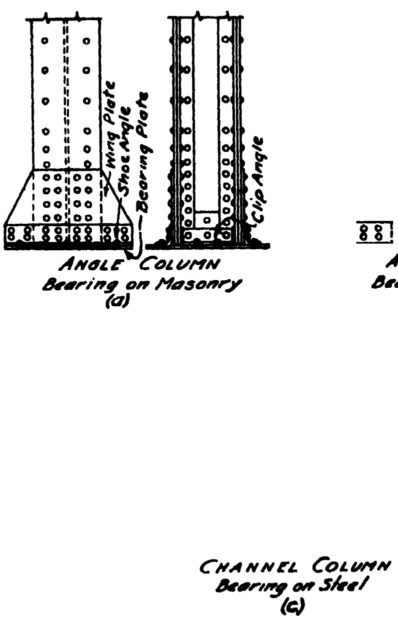


Fig. 414

It should be understood that the above classification of loads refers only to the range of loads for which built-up steel bases may be used. That is, a base plate thicker than 1¼″ is seldom employed, — rolled steel slabs being used instead. The sizes given for the "medium load" represent about the customary built-up base.

After the plan dimensions of the base plate have been established, the next step in the design is to determine the thickness of the plate. One of the critical places is the projection beyond the column, *b*, as shown in Fig. 415 (*a*). The action is that of a cantilever about the plane *a*–*a*, at the toe of the fillet of the shoe angle. This is conveniently analyzed by considering a strip 1″ wide, as shown. The load per linear inch on the cantilever is then the pressure per sq. in. on the base, *p*, assuming a uniform distribution. The external bending moment is thus $M_e = p \cdot b^2 \div 2$. The size of the angle is assumed (see classification above). The combined thickness of the angle and the plate may be counted upon as acting as one thickness, — providing that the angle is riveted to the plate with a sufficient number of rivets to resist the horizontal shear at the plane of contact of the angle with the plate. If the customary spacing of 3″ o.c. is used, there generally

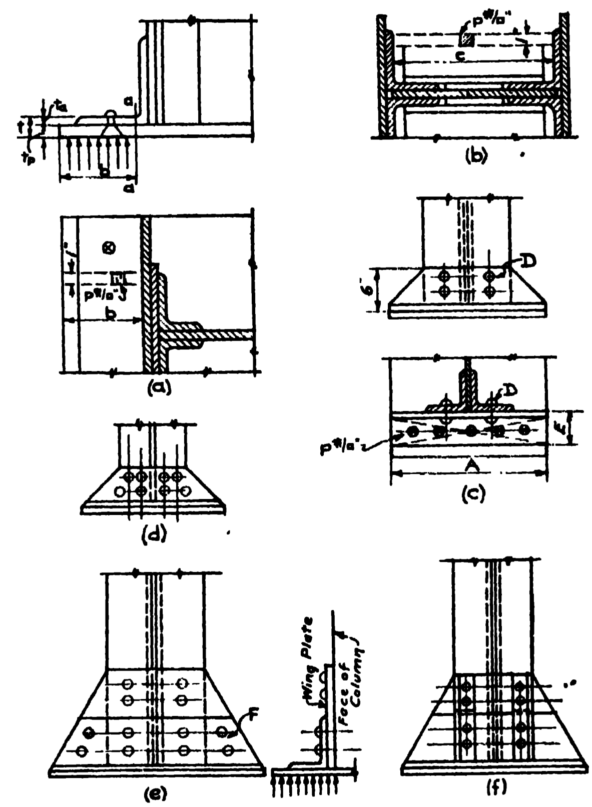


Fig. 415

will be ample resistance to such shear. The total required thickness, *t* in Fig. 415 (*a*), may be determined from

$$M_e = \frac{s \cdot 1 \cdot t^2}{6} \quad \text{or } t = \sqrt{\frac{6\,M_e}{s}} \quad (1) \text{ in which}$$

s = the maximum allowable fibre stress in #/□″.

If the thickness of the angle, t_a, is known, the thickness of the plate required is $t - t_a = t_p$.* If t_p exceeds say $1\frac{1}{4}''$ as a limit, then the angle thickness would have to be increased. If the thickness of the angle required under such conditions were excessive ($\frac{1}{8}''$ thicker than the diameter of the rivets or the maximum thickness rolled for that size of angle is the limit), then a rolled steel slab (Art. 262) or some other form of base would have to be used. Thus it may be seen that there is a limiting column load which may be carried safely by an ordinary built-up steel base, for a given set of conditions.

The thickness of the base plate, between the column flanges, should be checked. This may be tested for a 1″ strip, as shown in Fig. 415 (*b*), assuming it to be a fixed end beam. The moment is then $M_e = p \cdot c^2 \div 12$, from which the thickness may be determined from formula (1) above. Usually, the thickness of plate, as determined by the cantilever, controls, however. The clip angles, attached to the web and the base plate, help to stiffen the latter and thus it is not in simple flexure.

Another feature which should be investigated is the stress in the rivets which connect the shoe angles to the column flanges, or those at D in Fig. 415 (*c*). There should be enough rivets to develop the load on the portion of the base beyond the edge of the column, or in Fig. 415 (*c*), $p \cdot A \cdot E$. For ordinary, small columns, the maximum number of rivets is 4, where a 6 × 4 angle is used and only 2 gauge lines are available in the column face, as shown. If 4 gauge lines were available, as shown in (*d*), then 8 rivets might be counted upon. These of course must be figured at their controlling value, usually single shear. If the load on the area $A \cdot E$, in Fig. 415 (*c*), exceeds the strength of the rivets which may be driven in the upstanding leg of the shoe angle, then it becomes necessary to introduce a **wing plate.** This is illustrated in Fig. 415 (*e*). This plate allows the driving of more connecting rivets, and the vertical extent of the plate depends upon the number required. Rivets should be used at F to keep the angle and wing plate tied together, and may be counted upon to resist the load. They also help to keep the wing plate from buckling.†

Illustrative Prob. 261a. Design a steel built-up base to carry a load of 250,000# which is to rest upon a concrete footing. Column make-up: 12 × $\frac{3}{8}$ web plate, 4 ∟ 6 × 4 × $\frac{7}{16}$. Allowable pressure on the concrete = 500#/□″.‡

$$\text{Required area} = \frac{250,000}{500} = 500\square''.$$

$\sqrt{500} = 22+''$. Try base plate 24″ square.

$$\text{Actual pressure} = \frac{250,000}{24 \times 24} = 438\#/\square''.$$

Try 6 × 4 shoe angles (see Fig. 416 (*a*)).

$$M_e = \frac{p \cdot b^2}{2} = \frac{438 \times (4.75)^2}{2} = 4960''\#$$

$$t = \sqrt{\frac{6\,M_e}{s}} = \sqrt{\frac{6 \times 4960}{16,000}} = 1.28''$$

$\frac{1}{2} + \frac{3}{4} = 1.25''$ O.K. practically. Use 6 × 4 × $\frac{1}{2}$ shoe ∟
24 × $\frac{3}{4}$ × 2′-0″ Pl.

Test plate between column flanges (Fig. 416 (*b*)).

$$M_e = \frac{p \cdot c^2}{12} = \frac{438 \times (11.62)^2}{12} = 4930''\#$$

(O.K. — less moment than above).

Load outside of column face (see Fig. 416 (*c*)).

5.75 × 24 × 4.38 = 60,300#.

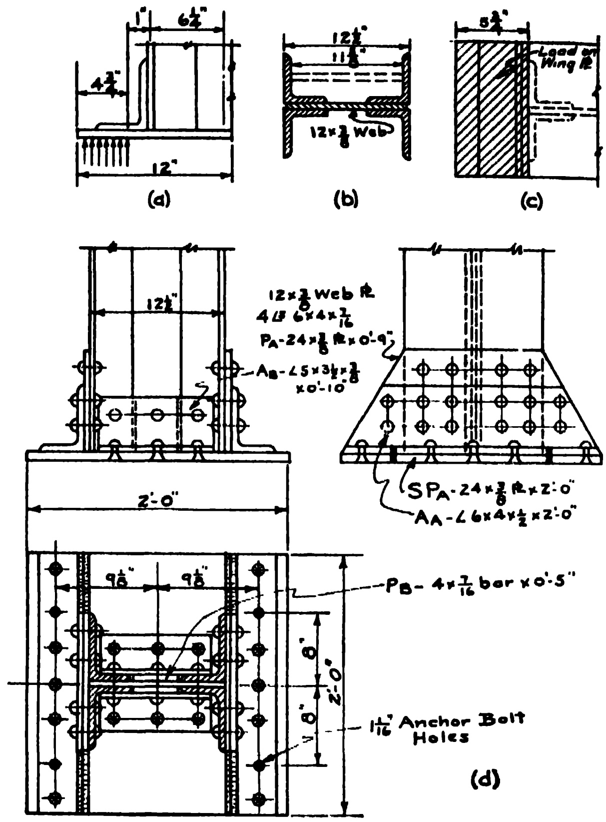


Fig. 416

Single shear, $\frac{3}{4}$ rivets = 5300#.
4 gauge lines available, or 8 rivets in ∟.
8 × 5300 = 42,400#. Hence wing plates are required.

$$\frac{60,300}{5300} = 11.3. \quad \text{12 rivets required.}$$

Wing plate must take up 1 row of rivets.
Make Pl. $\frac{5}{8}''$ thick.
Use 5 × $3\frac{1}{2}$ × $\frac{3}{8}$ clip ∟.

Fig. 416 (*d*) shows a detail of the base.

Prob. 261b. Design a built-up steel base to carry a load of 400,000# if the allowable bearing is 600#/□″. Column make-up: 14 × $\frac{3}{8}$ web plate, 4 ∟ 6 × 4 × $\frac{7}{16}$, and 2 flange plates 14 × $\frac{1}{2}$.

* If the angle were not connected to the base plate, or there were an insufficient number of rivets to resist the horizontal shear discussed, the method of design would have to be altered. The moment of resistance of the angle (based upon its thickness) would have to be subtracted from the total moment, M_e, and the thickness of the base plate would be proportioned to carry the remainder of the moment.

† Some detailers use stiffener angles along the sloping edges of the wing plates to prevent them from buckling when the wing plates are very large, but this is inadvisable and usually unnecessary, and complicates the details. Stiffener angles such as shown in Fig. 415 (*f*) are also cumbersome.

‡ Although the column would be set in grout, the concrete directly underneath it would control.

Prob. 261c. Design a built-up steel base to carry a load of 290,000# if column is composed of 2 — 12″ channels 20.7# per ft., and 2 — 14 × $\frac{3}{8}$ plates. Channels turned out and are $7\frac{1}{2}$″ back to back. Allowable bearing 500#/□″. Draw the details at a scale of $1\frac{1}{2}$″ = 1′-0″.

262. Rolled Steel Slabs.*

When a built-up steel base is not sufficient to distribute the column load upon the footing, the rolled steel slab may be used. Some designers prefer to use them for columns of any importance, as the details for the end of the column are simpler. A pair of standard 6 × 4 angles may be riveted to the column, as shown in Fig. 417 (*a*), to seat the

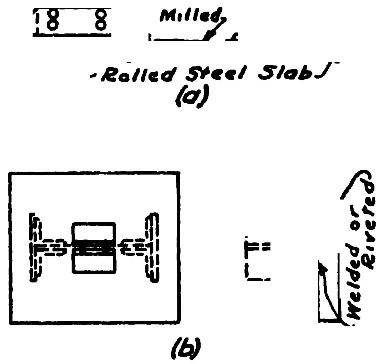


Fig. 417

shaft, or a pair of clip angles may be attached to the rolled steel slab, as shown in (*b*).† The latter are principally for the purposes of erection, and aid in setting the column at the proper point. In some cases, these angles are welded to the slab, and in others, they are riveted. If welded on, there is a danger that they will get knocked off before the columns are in place, but riveting them is awkward and generally they are welded. Figure 418 shows a group of rolled steel slabs. A disadvantage is that many structural shops do not carry many rolled steel slabs in stock, so that often they must be ordered from the mill. This may delay the shipment of the steel to the job to some extent.

The plan dimensions of a rolled steel slab are established by the bearing requirements. If the slab is to rest upon a concrete footing, a bearing stress of 500#/□″ for the concrete may be used. The slab is usually grouted into place, but the bearing of the concrete under the grout is the governing factor. When a slab is to rest upon a steel grillage footing, usually a layer of $\frac{1}{2}$″ to $\frac{3}{4}$″ of grout is placed between the base and the top tier of the grillage. It would be a relatively simple matter to

Fig. 418

plane the bearing surface of the slab, but it would involve considerable work and expense to plane the top flanges of the beams in the upper tier of the grillage, where the rolled steel slab would rest. Hence the layer of grout is used to give a level bearing. In usual cases, the allowable bearing for neat, Portland cement grout is 1000#/□″, but when neat grout between steel, not over $\frac{1}{2}$″ thick, is used, 1500#/□″ may be specified.‡ When the required bearing area is known, the plan dimensions of the slab should be fixed to the nearest 2″ above the requirements. These should naturally conform to the column section above, as far as possible. The slab should be square, preferably, but rectangular slabs are often used in order to keep the projections in each direction as nearly equal as possible, and thereby keep the thickness of the slab a minimum.§

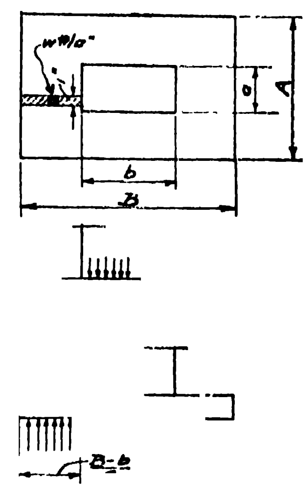


Fig. 419

The thickness of the slab must be sufficient to resist the bending induced in it. The method of calculating the moment is varied according to one's judgment of how the bending takes place. Some believe that the maximum moment occurs at the center-line of the column, and others base the calculations on the projection of the slab beyond the face of the column, acting as a simple cantilever. If the following symbols are assumed in Fig. 419:

* Rolled steel slabs may be used for cases where it is desirable to "turn" a column in an upper story, that is, so that the web of the upper section is normal to that of the lower section. This is sometimes done to allow heavy girders to be connected to the flanges instead of to the webs. The thick rolled steel slab is designed in a manner similar to that for bases.

Another place where rolled steel slabs are used is to seat columns on top of supporting girders, or to seat girders on top of columns. The columns should be anchored through the steel slabs in such cases.

† Some engineers prefer to use both pairs of angles. In any instance, rolled steel slabs are shipped loose to the job and are not attached to the column shaft before shipment. This allows the contractor to set the slabs in their proper positions before the other steel arrives.

‡ Recommended by the National Board of Fire Underwriters.

§ Other factors may also dictate that the slab be rectangular.

w = the actual* intensity of pressure on the base in #/□″,
B = the length of the rolled steel slab in a given direction in ins., and
b = the outside dimension of the column in the corresponding direction, in ins.,

then the bending moment, M_e, for a strip 1″ wide, may be expressed by

$$M_e = \frac{w(B^2 - b^2)}{8} \text{ (about the center-line of column),}\dagger \qquad (S\text{–}71)$$

or,

$$M_e = \frac{w(B - b)^2}{8} \text{ (as a cantilever).}\dagger \qquad (S\text{–}72)$$

The two above expressions are not equal in value, and the first is larger than the second. The cantilever method (formula (S–72)) is more commonly used, — it being reasoned that the column above the slab is not as "flexible" as a beam flange on a bearing plate might be, as the column has a great depth in the direction of the bending in this case. In the above formulas, the dimensions used should be the ones which involve the maximum projection. Some designers also calculate the bending moment for a 1″ strip between the flanges of the column, assuming it to act as a fixed end beam $\left(M = \frac{w \cdot L^2}{12}\right)$. This moment usually will not control, however. When the value of the maximum bending moment has been obtained, the thickness of the slab may be determined from

$$t = \sqrt{\frac{6\ M_e}{s}}, \text{ in which}\dagger$$

t = the required thickness of the rolled steel slab in ins.,
M_e = the maximum bending moment in in.-lbs., and
s = the maximum allowable fiber stress in #/□″, usually 16,000.

In general, the thickness should be a multiple of ½″.

Illustrative Prob. 262a. Determine the size of a rolled steel slab to distribute the load of 640,000# for the column shown in Fig. 420. Concrete footing.

$$\text{Bearing area required} = \frac{640{,}000}{500} = 1280□''$$

$\sqrt{1280} = 35.7+''$ Use 36″ × 36″ slab.

$$\text{Maximum projection} = \frac{36 - 14}{} \quad 11''$$

$$w = \frac{640{,}000}{36 \times 36} = 493\#/□''$$

$$M_e = \frac{w\,(B - b)^2}{8} = \frac{493 \times (22)^2}{8} = 29{,}900''\#$$

$$t = \sqrt{\frac{6 \times 29{,}900}{16{,}000}} = \sqrt{11.22} = 3.49''$$

Use slab 3½″ thick.‡

Clear distance between flanges = 12½ − 2 × ⅜ = 11.75″

$$M_e = \frac{w \cdot L^2}{12} = \frac{493 \times (11.75)^2}{2} = 5460''\# \text{ (does not control)}$$

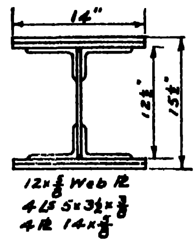


FIG. 420

Illustrative Prob. 262b. If the conditions in Illustrative Prob. 262a were the same except that the slab were to rest upon a steel grillage footing, what size of rolled steel slab would be required? Use ½″ grout maximum.

Maximum allowable bearing stress = 1500#/□″

$$\text{Bearing area required} = \frac{640{,}000}{1500} = 427□''$$

$\sqrt{427} = 20.7''$ Try 24″ × 24″ slab.

Assuming 6 × 4 bracket angles for the flanges, as in Fig. 417 (a),

15½ + 4 + 4 = 23½″. 24″ minimum dimension.

$$w = \frac{640{,}000}{24 \times 24} = 1110\#/□''$$

$$\text{Maximum projection} = \frac{24 - 14}{2} = 5''$$

$$M_e = \frac{1110 \times (10)^2}{8} = 13{,}900''\#$$

$$t = \sqrt{\frac{6 \times 13{,}900}{16{,}000}} = 2.29''$$

Use 24″ × 2½″‡ × 2′-0″ slab.

If a rectangular slab were used, the 24″ dimension across the flanges must be used, and $\frac{427}{24} = 17.8$, say 18″ could be employed for the other dimension, leaving only a 2″ projection. The maximum projection is then $\frac{24 - 15.5}{2} = 4.25''$.

$$w = \frac{640{,}000}{24 \times 18} = 1482\#/□''$$

$$M_e = \frac{1482 \times (8.5)^2}{8} = 13{,}420''\#$$

$$t = \sqrt{\frac{6 \times 13{,}420}{16{,}000}} = 2.24''$$

A 24″ × 2½″ × 1′-6″ slab could be used, but under usual conditions the square slab would work out better with the footing design.

Rolled steel slabs are available up to and including a 12″ thickness, by ½″ increments, but ordinarily 9″ is a usual mill-stock limit, and a 6″ thickness is sometimes considered an economical limit. An-

* The actual intensity of pressure is the load divided by the actual area of the base (not the allowable pressure, by which the size of the base was established). In this way, some thickness of plate may be saved in many cases.

† For the derivation of these formulas, refer to Art. 15.

‡ This is the finished thickness and planing must be allowed for (see following discussion).

other type of base may be cheaper than using too thick a slab. Slabs 4″ thick or less, and up to 36″ wide, may be straightened true and smooth in a hydraulic press. For thicknesses exceeding 4″, a surface must be planed if it is to rest on steel. A surface which is to bear on concrete need not be planed, as the slab is set in grout. An allowance of ½″ should be made in the thickness for each planed surface. **Thus for thicknesses greater than 4″, rolled steel slabs are ½″ less than the ordered thickness when they are to bear on concrete or grout, and 1″ less when they are to bear on steel.**

In order to speed up delivery from the mill, standard commercial sizes are now adopted by some companies. These are rolled in quantity, stocked, and then cut to the lengths ordered, so that less time is required to obtain them. Any reasonable length is obtainable. The following is a standard list:

Rolled Commercial Sizes*

14 × 1½, 16 × 1½, 20 × 2, 24 × 2½, 24 × 3, 28 × 3, 32 × 3½, 30 × 4, 36 × 4, 34 × 5, 40 × 5, 34 × 5½, 44 × 5½, 34 × 6, 34 × 6½, 42 × 6½, 34 × 7, 42 × 7, 40 × 7½, 34 × 8, 48 × 8, and 48 × 9.

Prob. 262c. Determine the size of a rolled steel slab to distribute a column load of 750,000# on a concrete footing. Column make-up: 14 × ½ web plate, 4 — 6 × 4 × ½ angles, 2 — 14 × ⅝ cover plates. Use a commercial size.

Prob. 262d. Design a rolled steel slab to distribute a load of 480,000# on a grillage footing. Column make-up: 12 × ⅜ web plate, 4 — 5 × 3½ × ⅜ angles, 2 — 12 × ½ cover plates.

263. Multiple Steel Slabs.

When the foundation is good for a high bearing pressure, such as hard rock, a series of rolled steel slabs, similar to that illustrated in Fig. 421,† may

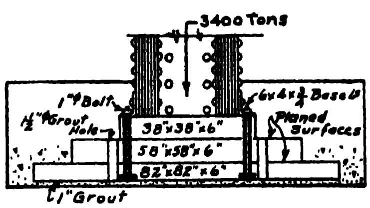


FIG. 421

be used. In many cases, it may work out cheaper than some other form of footing, because only a small depth is required. This is true especially when the required length of grillage beams is 3′-0″ or less on a concrete footing, as excavation and blasting are saved. The design of each slab is similar to that given in the preceding discussion (Art. 262).

* As listed in "Design of Structural Steel for Tier Buildings," edited and copyrighted by the American Bridge Company, New York City.

† Based on an article by Mr. E. W. Stern, Consulting Engineer, New York City, in Engineering News-Record, May 4, 1922.

264. Cast-iron Pedestals.

SPECIFICATION CLAUSE‡

> Cast-iron bases or shoes shall be planed on top. Bases which rest on steel girders shall be planed top and bottom. The thickness of metal shall be not less than 1 inch. The inclination of the outer edge of the ribs with the horizontal shall be not less than 45 degrees. Whenever one side of the bed plate exceeds 3 feet in length a reinforcing flange at least 3 inches high shall be provided.

For columns carrying very heavy loads, the thickness of a steel or cast-iron plate, or of a rolled steel slab, may become impracticable, and sometimes a cast-iron pedestal is used instead as a load may be distributed in a relatively small height. This is sometimes the case in relatively high buildings. One advantage is that the details at the bottom of the column are simple. Bracket angles are riveted to the column, which in turn are attached by bolts through the pedestal.§ Figure 413 (*b*) illustrates the detail, and Fig. 422 shows an actual installation.

FIG. 422

Pedestals are generally made square in plan at the base,¶ and either rectangular or square in plan at the top. Figure 423 gives a typical illustration. The size of the bottom is governed by the area required for the bearing. The minimum size of the top is determined by seating the bracket angles of the column and providing space for the bolts. The height, *h*, is usually made from ⅓ to ½ the width of the base. When a "hub" type is employed, as in Fig. 423 (*b*), the diameter of the hub must be assumed. This is often made about 8″ inside, or to conform to the column above, and the thickness of the metal is commonly made 1¼″ minimum. The top plate is made 1″ minimum, and 1¼″ is a customary value. The thickness of the base plate is about 1½″ minimum, and 1¾″ represents an average. The number of ribs is a matter of judgment, but they should be about as shown in Fig. 423 (*b*) for bases 2′-0″ to 4′-0″ square, and about as shown in Fig. 424 for bases 5′-0″ to 6′-0″ square. The thickness should be 1″ minimum. In fact, no metal should be less than 1″ thick for this work, and not more than 4″ thick, and preferably 3″. The corner ribs should be from ¼″ to ½″ thicker than the others, as they are longer. The thickness of any rib should not be less than $\frac{1}{18}$ the clear height between the top and

‡ From the Building Code of the National Board of Fire Underwriters, New York City.

§ Rivets are never used through cast iron. Anchor bolts may be taken through the pedestal if necessary for uplift, although this is unusual.

¶ Sometimes pedestals are made round, as they are applied better to circular piers or caissons. In such cases, the bending moment may be approximated as the load times 0.10 the diameter of the base plate.

bottom plates. In summary, an effort should be made to have the thicknesses of all the parts somewhere near the same. If radically different thicknesses were used for the component parts, cracks in the casting might develop, due to the different rates of cooling. For this reason, it is wise to avoid having the ribs meet under the corners of the top plate, and hence a " hub " type of pedestal is advantageous.

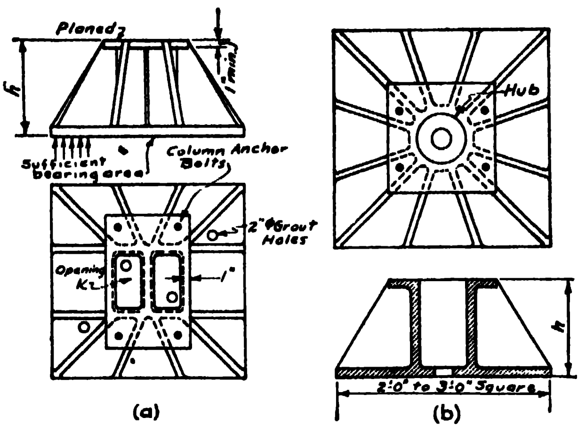


FIG. 423

Fillets should be used at all intersecting traces of the parts, as they aid in strengthening the casting and in its manufacture. The radius of the fillets is usually $\frac{1}{2}''$. The top of the pedestal is planed to provide a level seat for the column. The holes for the column anchor bolts are not cored in the casting, but are drilled later ($\frac{1}{16}''$ greater than diameter of the bolts) so that accurate setting of the column is possible. Grout holes are provided in the bottom plate (usually $4 - 2''\phi$, as shown). On the job, the pedestal is leveled on top of the footing by small steel wedges, then formed around, and grout is poured until it appears in all of the holes, in order to bed the pedestal properly. No two grout holes should be on the same line parallel to either axis of the base. When the grout appears in the holes so located, it gives reasonable assurance that the grout is in a complete layer under the pedestal. Two grout holes should be near the center of the pedestal. Large openings should occur in a type such as " K " in Fig. 423 (*a*), or else be formed by the space inside the hub, as in (*b*). This is essential, so that the cores in the casting process may be removed. If the required thickness of the base becomes excessive, or the thickness otherwise established is not strong enough to resist the local bending between the ribs, a rim (or border) should be used, as shown in Fig. 424. In fact, a rim should be used on all large pedestals as a matter of protection, especially those larger than 4'-0" square.

Forms other than those shown may be devised and have been used. The top plate may be solid and round holes placed in the ribs to extract the core. In others, the middle ribs may be made to form a circle, with one rib across, to carry the web of the column. In some types, the metal of the column above may not bear accurately over the ribs. This would cause bending in the top plate and should be avoided.

In designing a cast-iron pedestal, the stresses cannot be computed accurately, and hence a high factor of safety should be used, as well as to provide for the tendency of cast iron to have flaws and to be unreliable. The section is established mainly by the use of the flexure formula. The design must be more or less " cut and try," because of the many variables. The minimum sectional area may be calculated with an assumed set of dimensions (not including the ribs). The neutral axis of this section may be located, and the resulting moment of inertia computed. This will allow testing the maximum flexural stress. The shear should also be investigated. There should be enough sectional area in the ribs and hub at a plane just below the top plate to resist the load on the pedestal. The thickness of the bottom plate must be sufficient to resist the local bending between the ribs. Common working stresses for cast iron are 3000#/□" for both tension and shear. Cast iron is a good material in bearing and compression, so that the bearing of the column on the top plate, and the compressive fiber stress due to bending, do not usually have to be investigated.

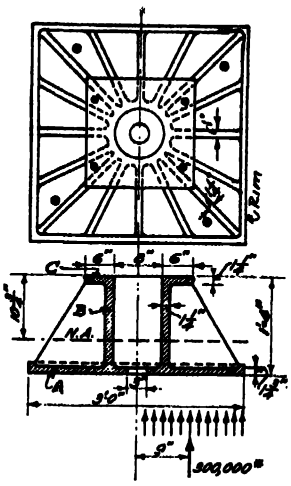


FIG. 424

Illustrative Prob. 264a. If the column load in Fig. 424 is 600,000#, determine if the section of pedestal shown is satisfactory. Column 10" in size.

Assume 6" × 4" base angles (long legs vertical) 10 + 4 + 4 = 18". Allowing for overrun, etc., make top 20" square.

$$\text{Required bearing area} = \frac{600{,}000}{500^*} = 1200\square''$$

$\sqrt{1200} = 35.6''$ Make base 36" square.

Assume top $1\frac{1}{4}''$ thick, base $1\frac{3}{4}''$ thick, and hub $1\frac{1}{4}''$ thick. $36 \times \frac{1}{3} = 12''$. Make depth = 16".

The shaded area indicated in Fig. 424 is available, neglecting the fillets and rim.

$$\begin{aligned}
\text{Area "A"} &= \phantom{2\times{}}33 \times 1.75 = 57.75\\
\text{2 Areas "B"} &= 2 \times 13 \times 1.25 = 32.50\\
\text{2 Areas "C"} &= 2 \times 6 \times 1.25 = 15.00\\
\text{Total area} &= 105.25\square''
\end{aligned}$$

Taking moments about the base,

$$\begin{aligned}
57.75 \times 0.87 &= 50.53\\
32.50 \times 8.25 &= 268.12\\
15.00 \times 15.38 &= 230.62\\
\text{Sum} &= 549.27
\end{aligned}$$

$$\frac{549.27}{105.25} = 5.21'' = \text{distance of N.A. above base.}$$

Calculating the moment of inertia, I_0,

	I	+	$A \cdot d^2$
Area " A "	15	+	57.75 × (4.33)² = 1098
2 Areas " B "	458	+	32.50 × (3.04)² = 759
2 Areas " C "	2	+	15.00 × (10.16)² = 1550

$$I_0 = 3407''^4$$

Moment of resistance,

$$M_r = \frac{s \cdot I}{c} = \frac{3000 \times 3407}{5.21} = 1{,}940{,}000''\#.$$

External moment, $M_e = 300{,}000 \times 9 = 2{,}700{,}000''\#$.

The selected cross-section is **not satisfactory,** and a thicker top and bottom plate must be assumed.

Local bending between ribs.

* If an assumed pedestal does not test out satisfactorily, of course a larger one must be used. The most effective places to add area are in the top and bottom plates.

Average distance center to center of ribs = 4″±.
Clear distance between ribs = 4″ − 1″ = 3″.

Plate continuous. End span $= \dfrac{w \cdot L^2}{12} = 1.0\, w \cdot L^2$ in.-lbs.

Actual pressure on base $= \dfrac{600{,}000}{36 \times 36} = 464\#/\square''$.

Consider a 1″ strip.

$$M_c = 1.0 \times 464 \times (3)^2 = 4180''\#$$

$$4180 = \frac{3000 \times 1 \times t^2}{6} \cdot t^2 = 8.36 \quad t = 2.9, \text{ say } 3'' \text{ thick.}$$

Sectional area of hub = $(9.25)^2 \times \pi \times 1.25 = 336\square''$.
12 ribs 1″ thick, and 4 ribs 1¼″ thick, average length under top plate = 7″ each

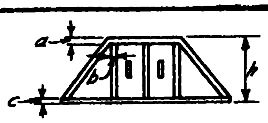
$$12 \times 1 \times 7 + 4 \times 1\tfrac{1}{4} \times 7 = 133\square''$$

Shear resisting area = 336 + 133 = 469, say 470□″

$$\frac{600{,}000}{470} = 1280\#/\square'' \text{ shear O.K.}$$

The shear at the edge of a local rib on a strip of the bottom plate should also be tested.

The design of a cast-iron pedestal is laborious work, and in many cases, attempts are made to standardize them. This will also allow the re-use of patterns and hence decrease the cost of the pedestals considerably. Figure 425 gives a typical illustration of this kind. Naturally the number of sizes of pedestals for a given job should be limited, and the column loads to be carried may be grouped.



Pressure on Masonry = 30 Tons per sq. ft.
Tension for C.I. = 4000 lbs. per sq. in.
Shear for C.I. = 3000 lbs. per sq. in.

Load on Base. Tons	Size of Base	Weight of Base	h	a	b	c
100	22″x22″	360	7″	1⅛″	1″	1½″
125	25″x25″	440	7″	1⅛″	1″	1½″
150	27″x27″	570	8″	1¼″	1″	1⅝″
175	29″x29″	650	9″	1¼″	1″	1⅝″
200	31″x31″	770	10″	1¼″	1″	1⅝″
250	34″x34″	1050	12″	1¼″	1⅛″	1¾″
300	38″x38″	1300	14″	1¼″	1⅛″	1¾″
350	41″x41″	1600	14″	1¼″	1⅛″	1⅞″
400	44″x44″	1950	16″	1⅜″	1¼″	2″
450	46″x46″	2300	16″	1⅜″	1¼″	2⅛″
500	49″x49″	2600	18″	1⅜″	1¼″	2¼″

FIG. 425*

* Reproduced with permission, from data of Mr. Francis W. Wilson, Consulting Engineer, Cambridge, Mass.

Prob. 264b. Establish the proper thicknesses of the bottom plate in Illustrative Prob. 264a if the top plate is made 1¼″ thick.

Prob. 264c. Design (make the calculations) for a base plate similar to Fig. 424 to carry a load of 800,000#. Assume column 12″ in size. Make height 1′-8″.

Prob. 264d. Check the size of pedestal shown in Fig. 425 for a load of 250 tons.

265. Cast-iron Plates.

For column loads which do not require as heavy a base as a cast-iron pedestal, one form of alternate which may be used is a cast-iron plate, similar to that shown in Fig. 426. The design is similar to that discussed and illustrated for cast-iron pedestals (Art. 264). The maximum overall thickness should be limited to 4″. In general, it may be said that, for such cases, rolled steel slabs would be preferable and more reliable, and probably less expensive.

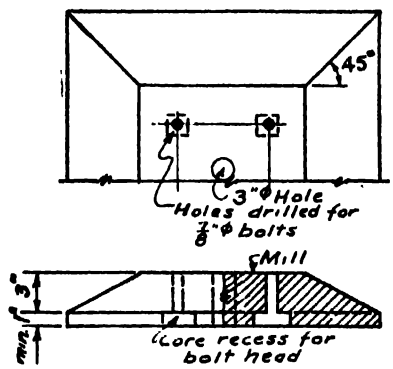


FIG. 426

266. Anchorage.

The usual column is often not tied into the footing with anchor bolts, such as in office buildings of ordinary heights, apartment houses, and the like. However, when columns are subjected to wind uplift, or when direct uplift is possible, such as in cantilever, theatre balcony framing, anchor bolts should be used. In mill buildings which are subjected to shocks and vibration, the columns should be anchored to the footings in the usual case. It is considered advisable to provide anchor bolts in all cases in order to protect the building against unusual conditions such as for earthquakes, tornadoes, and the like.

A check should be made of anchorages where sliding is possible, especially in places where earthquake disturbances are frequent. This sort of movement causes shear at the plane where the column base rests upon the footing. The shear induced by the average quake may be expected to average about 50% of the load, inasmuch as the incipient sliding is a function of the inertia of the column load. A definite portion of this may be carried by the frictional resistance between the two materials, which again is the load multiplied by the

coefficient of friction between the two materials. As the uniformly distributed load on concrete footing is approximately 500#/sq. in. and the coefficient of friction between steel and concrete is 0.35, the resistance to sliding is about 175#/□″. The balance must be provided by shear resistance in the anchor bolts.

If the uplift may be calculated as a definite value, then by dividing it by 16,000#/□″, the required net sectional area of the anchor bolts may be obtained. By establishing the number of bolts (usually either 2 or 4), and using a table of the net areas for screw threads (Table 78), the size of the bolts may be obtained. The anchorage of the bolts may be established by using either of two methods, namely:

(1) To embed the bolts for a length such that the safe tensile strength of each will be developed by the bond with the concrete, or

(2) To use washers at the lower ends of the bolts, of such a size that the bearing of the washers on the concrete will be equivalent to the tensile strength of the bolts.

In the first method, the bond is the adhesive strength between the surfaces of contact of the bolt with the concrete (circumference of bolt times the length of embedment). A safe bond stress may be taken as 80#/□″. If the following symbols are used,

a_s = the cross-sectional area of one bolt, in sq. ins.,
f_s = the allowable tensile stress in the bolt in #/□″, usually 16,000,
x = the length of the imbedment in ins.,
i = the diameter of the bolt in ins., and
u = the allowable bond stress in #/□″ of surface contact,

the allowable tension in the bolt is

$$T = a_s \cdot f_s = \frac{\pi \cdot i^2}{4} \cdot f_s.$$

This must be developed by the bond resistance, or

$$\frac{\pi \cdot i^2}{4} \cdot f_s = (\pi \cdot i) \cdot x \cdot u, \text{ or}$$

$$\therefore \quad x = \frac{f_s}{4\,u} \cdot i \qquad (S\text{-}73)$$

For f_s = 16,000#/□″, and u = 80#/□″,

$$x = \frac{16{,}000}{4 \times 80} \cdot i = 50\,i$$

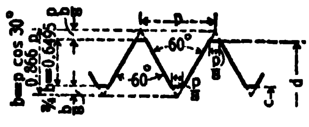
or 50 diameters of the bolt. Sometimes rods, threaded at the column base end, are used, and hooked at the lower ends. Right angle hooks are of no real benefit but the length of such a hook may be counted upon as length of embedment. Semicircular hooks add to the bond, and the length of the hooked portion may be considered as equal to twice its real length as embedment. There should be sufficient concrete engaged by the hook to prevent the latter from straightening out.

It is not always convenient to embed a rod into a footing 50 diameters, and bolts are not usually made in such lengths, so that **the more common method of anchorage** involves the use of washers. This is also the more positive and direct method. Several ways of providing bearing at the ends of the bolts may be used, such as shown in Fig. 427. A small plate may be used for each bolt, or a pair of bolts may be jointly anchored by a bar engaging the two. The thickness of the plate must be sufficient to resist the local bending on it. The depth at which the washer is placed must be such that the shearing

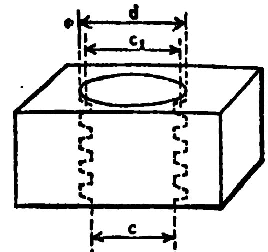
TABLE 78*

SCREW THREADS

Bolts, Rods, Eye Bars, Turnbuckles, Sleeve Nuts and Clevises



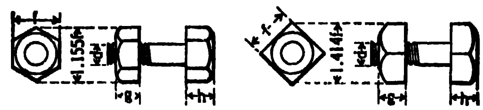
(a)



(b)

BOLT HEADS AND NUTS

AMERICAN BRIDGE COMPANY STANDARD



Rough Nut		Finished Nut		Rough Head		Finished Head	
f	g	f	g	f	h	f	h
1.5d+1/8″	d	1.5d+1/16″	d−1/16″	1.5d+1/8″	0.5f	1.5d+1/16″	0.5f−1/16″

(c)

Diameter		Area		Number of Threads per In.	Diameter		Area		Number of Threads per In.
Total, d, In.	Net, c, In.	Total Dia., d, Sq. In.	Net Dia., c, Sq. In.		Total, d, In.	Net, c, In.	Total Dia., d, Sq. In.	Net Dia., c, Sq. In.	
1/4	.185	.049	.027	20	2 1/2	2.175	4.909	3.716	4
3/8	.294	.110	.068	16	2 5/8	2.300	5.412	4.156	4
1/2	.400	.196	.126	13	2 3/4	2.425	5.940	4.619	4
5/8	.507	.307	.202	11	2 7/8	2.550	6.492	5.108	4
3/4	.620	.442	.302	10					
7/8	.731	.601	.419	9	3	2.629	7.069	5.428	3 1/2
					3 1/4	2.879	8.296	6.509	3 1/2
1	.838	.785	.551	8	3 1/2	3.100	9.621	7.549	3 1/4
1 1/8	.939	.994	.693	7	3 3/4	3.317	11.045	8.641	3
1 1/4	1.064	1.227	.890	7					
1 3/8	1.158	1.485	1.054	6	4	3.567	12.566	9.993	3
1 1/2	1.283	1.767	1.294	6	4 1/4	3.798	14.186	11.330	2 7/8
1 5/8	1.389	2.074	1.515	5 1/2	4 1/2	4.028	15.904	12.741	2 3/4
1 3/4	1.490	2.405	1.744	5	4 3/4	4.255	17.721	14.221	2 5/8
1 7/8	1.615	2.761	2.049	5					
					5	4.480	19.635	15.766	2 1/2
2	1.711	3.142	2.300	4 1/2	5 1/4	4.730	21.648	17.574	2 1/2
2 1/8	1.836	3.547	2.649	4 1/2	5 1/2	4.953	23.758	19.268	2 3/8
2 1/4	1.961	3.976	3.021	4 1/2	5 3/4	5.203	25.967	21.262	2 3/8
2 3/8	2.086	4.430	3.419	4 1/2	6	5.423	28.274	23.095	2 1/4

* American Bridge Co. Std. from the Carnegie Pocket Companion.

strength of the concrete above it is ample.* The weight of the footing, plus any engaged earth, must be more than the uplift. This is usually more than safe. For large anchor bolts, pieces of channel, or angle, are used to provide sufficient bending resistance. When columns are arbitrarily anchored,

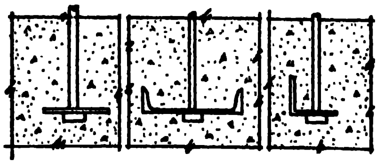


FIG. 427

the size of the anchor bolts is nominal, but not less than $2 - 1''\phi$ are generally employed. These should be set as far apart as possible. The washers and embedment are made sufficient to develop the strength of these bolts in such cases as well. The size of the holes in the column base is commonly made $\frac{5}{16}''$ larger than the diameter of the bolts, to allow for adjustment in setting the column. The bolts are set in the concrete by the use of a template, as illustrated in Fig. 428. (For a discussion of anchor bolt setting plans, see Art. 249.)

* For anchor bolts set in stonework, the holes are made about 1" larger than the diameter of the bolt, and the bolts are set in cement grout. This is more reliable than using either lead or sulphur, and the grout is easier to use, is stronger, and is a better preservative. The resistance in such cases may be calculated at 400#/□" of surface contact.

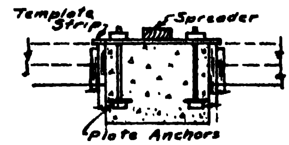


FIG. 428

Prob. 266a. If 4 anchor bolts are to be used, what size is required for an uplift of 50,000#? What size of bearing plates is required if the allowable bearing is 500#/□"?

Prob. 266b. For a wall column load of 196,000#, a soil bearing value of 6000#/□', and a safe bearing stress of 500 #/□" on the concrete, design a column base to resist earthquake action coupled with a possible uplift of 26,000#.

CHAPTER 23

MISCELLANEOUS COLUMNS

Section 23A

STEEL STRUTS

267. General.

In stair-framing, pent-houses, intermediate beam supports, and so on, large-sized columns are usually unnecessary, and for light loads and short lengths, angles may be used. These are often called struts. Figure 429 shows a typical shop detail of a single angle strut. The design of struts is similar to that for steel columns (Chap. 21), and, in fact, they are small steel columns and all the common requirements apply to them. Sometimes a pair of channels are used for heavier loads and where available space is limited as well.

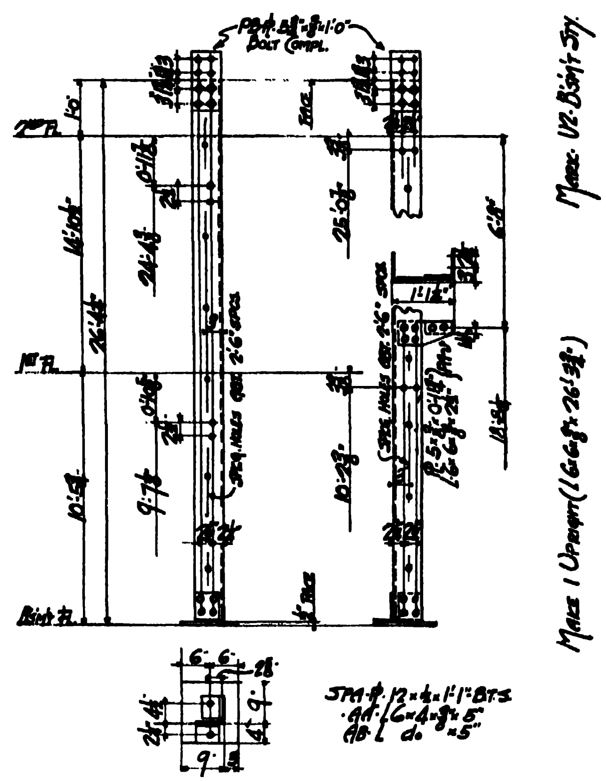


Fig. 429

Figure 430 shows common sections of angle struts that in (*b*) being most usual. The section in (*a*) is suitable for light loads, but is not economical of material, as the strength is limited by the least radius of gyration, which is about the 3-3 axis. A single angle strut is readily adaptable to corner framing, and for hangers where the section is in tension. Equal-legged angles are generally employed. When two angles are used, unequal-legged angles with the long legs vertical are common, as the radii of gyration about the 1-1 and 2-2 axes are more nearly balanced. The legs may be in contact or apart, depending upon the conditions.

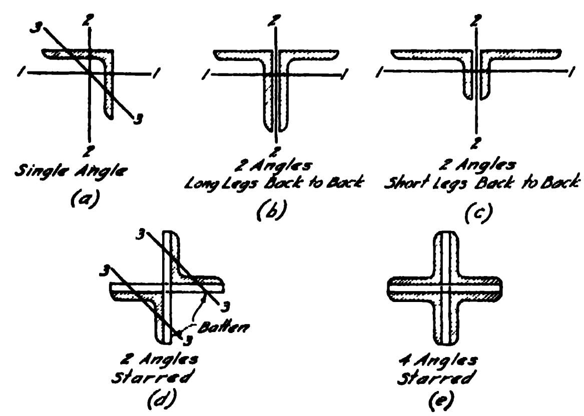


Fig. 430

The section in Fig. 430 (*c*) is used only when unequal ratios of slenderness in the two respective directions occur, or when clearances require such a detail. The section in (*d*) is confined principally to bracing, and to eave and end struts in mill buildings (see Index). It is not normally economical of material as the radius of gyration about the 3-3 axis controls. The section in (*e*) is used only in special cases, such as under theatre balconies, or as reinforcement in columns (Art. 276). An advantage in the latter instance is the fact that the circle of fire protection material, when circum-

scribed, is of a smaller diameter than when an H section is used. It is not particularly economical of steel.

Tables are used to a great extent in the design of struts. These give various radii of gyration, sectional areas, and in some cases, the allowable loads for various lengths.

268. Single Angle Struts.

The design of single angle struts is discussed in Art. 206. The important point to keep in mind is that the controlling radius of gyration is about the 3–3axis. This eliminates the smaller sizes of angles for any considerable lengths on account of the limiting ratio of slenderness. A safe guide in this respect is that **a 3 × 3 cannot be used for lengths over 6′-0″.**

Illustrative Prob. 268a. Design a single angle strut 9′-0″ long to carry a load of 35,000#.

$$\text{Minimum } r = \frac{9 \times 12}{120} = 0.9''.$$

From tables, 5 × 5 ∟ minimum, r_{3-3} (ave.) = 0.97″

$$p = 16{,}000 - \frac{70 \times 9 \times 12}{0.97} = 8230\#/\square''$$

$$A = \frac{35{,}000}{8230} = 4.25\square''\text{ reg'd.} \quad 5 \times 5 \times \tfrac{1}{2}\ ∟,\ A = 4.75\square''$$

$$w = 16.2\#/\text{ft.}$$

If room is available, a larger sized angle of thinner metal is more economical.

Try 6 × 6, $r_{3-3} = 1.19''$

$$p = 16{,}000 - \frac{70 \times 9 \times 12}{1.19} = 9650\#/\square''$$

$$A = \frac{35{,}000}{9650} = 3.63\square'' \qquad 6 \times 6 \times \tfrac{3}{8} \qquad A = 4.36\square''$$

$$w = 14.9\#/\text{ft.} \quad \textit{Use.}$$

A 6 × 4 × $\frac{3}{8}$ ∟ could be used but no gain is made.

Prob. 268b. Design a single angle strut 7′-0″ long to carry a load of 32,000#. Use equal legged angle.

Prob. 268c. Design a strut of a single unequal legged angle for the data of Prob. 268b. Compare the weights per ft. in the two examples.

269. Double Angle Struts (Long Legs Vertical).

The design of struts of two angles with the long legs vertical (in section) is discussed in Art. 206. The angles should be held together by stay rivets (with washers in between the angles, if separated) to make them act in unison. Theoretically, the ratio of slenderness of one angle between rivets should be less than that of the strut as a whole.

To illustrate for 2 ∟∟ 4 × 3 × $\frac{3}{8}$ for a 12′-0″ length, r_{3-3} = 0.64″ and r_{2-2} = 1.22″.

$$\frac{l}{r} = \frac{8 \times 12}{1.22} = 78.7$$

spacing of rivets, s. $\dfrac{s}{0.64} = 78.7$, or $s = 50.3''$.

One rivet in the middle of the length would be sufficient, but for practical reasons, a 2′-0″ spacing of rivets is generally used.

Illustrative Prob. 269a. Design a double angle strut to carry a load of 76,000#. Length 10′-0″. Use long legs vertical and $\frac{3}{4}$″ back to back of ∟∟.

$$\text{Minimum } r = \frac{10 \times 12}{120} = 1.0''.$$

From tables, minimum size ∟ = 3½ × 2½.
Usually a larger size will be more economical.
Try 5 × 3, r (ave.) = 1.30″

$$p = 16{,}000 - \frac{70 \times 10 \times 12}{1.30} = 9540\#/\square''$$

$$A = \frac{76{,}000}{9540} = 7.96\square''\text{ or }3.98\square''\text{ for 1 ∟.}$$

$$2\ ∟∟\ 5 \times 3 \times \tfrac{9}{16},\ w = 28.6\#/\text{ft.}$$

Several trials will be made to illustrate how the judgment is improved, and to show that larger sizes are more economical.

Try 5 × 3½, r (ave.) = 1.56″

$$p = 16{,}000 - \frac{70 \times 10 \times 12}{1.56} = 10{,}610\#/\square''$$

$$A = \frac{76{,}000}{10{,}610} = 7.16\square''\text{ or }3.58\square''\text{ for 1 ∟.}$$

$$2\ ∟∟\ 5 \times 3\tfrac{1}{2} \times \tfrac{7}{16},\ w = 24.0\#/\text{ft.}$$

Try 6 × 3½, r (ave.) = 1.43″

$$p = 16{,}000 - \frac{70 \times 10 \times 12}{1.43} = 10{,}130\#/\square''$$

$$A = \frac{76{,}000}{10{,}130} = 7.50\square''\text{ or }3.75\square''\text{ for 1 ∟.}$$

$$2\ ∟∟\ 6 \times 3\tfrac{1}{2} \times \tfrac{7}{16},\ w = 27.0\#/\text{ft.}$$

Try 6 × 4, r (ave.) = 1.67″

$$p = 16{,}000 - \frac{70 \times 10 \times 12}{1.67} = 10{,}970\#/\square''$$

$$A = \frac{76{,}000}{10{,}970} = 6.92\square'',\text{ or }3.46\square''\text{ for 1 ∟.}$$

$$2\ ∟∟\ 6 \times 4 \times \tfrac{3}{8},\ w = 24.6\#/\text{ft.}$$

The 5 × 3½ × $\frac{7}{16}$ ∟∟ are the most economical selection.

Prob. 269b. Design a strut of 2 ∟∟, long legs vertical, 10′-3″ long, to carry a load of 42,000#. Use $\frac{3}{8}$″ gusset plates.

270. Double Angle Struts (Short Legs Vertical).

When a beam frames into a strut at a level intermediate between the top and bottom, the short legs of a double angle strut are sometimes made the vertical ones. This gives a better balance of the ratios of slenderness in the two directions. The total load must of course be used in determining the required area (see Art. 206).

Prob. 270a. Design a strut, composed of 2 ∟∟, short legs vertical, 8′-0″ long, to carry a load of 28,000#. Use ½″ gussets.

271. Two Angles Starred.

Two angles "starred" as a strut are in reality two single angle struts tied together by batten plates to make them act in unison. The controlling radius of gyration of each angle is about the 3–3 axis, so that the value for the strut is twice that for one angle. The spacing of the batten plates should theoretically be such that the ratio of slenderness for one angle between batten plates is not greater than that for the

strut as a whole. Since the combined r is twice that for one angle, the spacing of batten plates is $\frac{1}{2}$ l. In practice, a spacing of 3′-0″ is generally used for the batten plates. The connections bringing the load on to such a strut must be such that they equally distribute the load to both angles, or the design must be altered accordingly.

Batten plates should be wide enough to allow 2 rivets at each end, and should be spaced 3′-0″ o.c.

Prob. 271b. Design a strut of 2 ∟, starred, to carry a load of 18,000# on an 11′-0″ length.

TABLE 79

RADII OF GYRATION										
SIZE		AREA			⅜ GUSSETS		½ GUSSETS		⅝ GUSSETS	
LEGS	THICK	2 ∟s	1-1	1-1	2-2	2-2	2-2	2-2	2-2	2-2
8 × 6	⅞	22.96	2.51	1.74	2.59	3.75	2.63	3.81	2.59	3.85
	¾	19.88	2.53	1.76	2.57	3.73	2.52	3.78	2.56	3.83
	⅝	16.72	2.54	1.77	2.46	3.71	2.50	3.76	2.84	3.81
	½	13.50	2.56	1.79	2.43	3.67	2.48	3.30	2.52	3.78
8 × 3½	⅞	18.60	2.53	0.87	1.36	4.16	1.41	4.21	1.46	4.25
	¾	16.12	2.55	0.88	1.33	4.13	1.39	4.18	1.43	4.23
	⅝	13.60	2.57	0.90	1.30	4.10	1.36	4.16	1.40	4.20
	½	11.00	2.58	0.91	1.30	4.00	1.36	4.12	1.37	4.18
7 × 3½	⅞	16.84	2.20	0.90	1.40	3.57	1.46	3.69	1.51	3.69
	¾	14.62	2.22	0.91	1.38	3.56	1.44	3.62	1.05	3.67
	⅝	12.34	2.24	0.93	1.37	3.55	1.41	3.60	1.46	3.64
	½	10.00	2.25	0.94	1.35	3.53	1.39	3.58	1.21	3.62
	⅜	7.60	2.27	0.96	1.33	3.50	1.37	3.55	1.41	3.59
6 × 4	⅞	15.96	1.86	1.11	1.78	2.96	1.76	3.02	1.80	3.06
	¾	13.88	1.88	1.12	1.71	2.94	1.73	2.99	1.78	3.04
	⅝	11.72	1.90	1.13	1.69	2.92	1.70	2.97	1.75	3.01
	½	9.50	1.90	1.15	1.67	2.90	1.68	2.94	1.72	2.99
	⅜	7.22	1.93	1.17	1.62	2.87	1.67	2.92	1.71	2.95
6 × 3½	⅞	15.10	1.87	0.93	1.49	3.05	1.54	3.10	1.59	3.15
	¾	13.12	1.89	0.94	1.46	3.02	1.50	3.05	1.55	3.12
	⅝	11.10	1.90	0.96	1.44	2.99	1.48	3.04	1.53	3.10
	½	9.00	1.92	0.97	1.41	2.97	1.46	3.02	1.50	3.08
	⅜	6.84	1.94	0.99	1.39	2.95	1.43	3.00	1.48	3.05
5 × 4	⅞	14.22	1.52	1.14	1.80	2.43	1.85	2.48	1.90	2.53
	¾	12.38	1.54	1.15	1.77	2.40	1.83	2.45	1.87	2.50
	⅝	10.46	1.55	1.17	1.75	2.38	1.80	2.43	1.85	2.48
	½	8.50	1.57	1.19	1.73	2.36	1.77	2.40	1.82	2.45
	⅜	6.46	1.59	1.20	1.70	2.34	1.75	2.38	1.80	2.43
5 × 3½	⅞	13.34	1.53	0.96	1.56	2.50	1.61	2.55	1.66	2.60
	¾	10.81	1.55	0.98	1.58	2.47	1.60	2.52	1.64	2.57
	⅝	9.84	1.56	0.99	1.49	2.44	1.58	2.50	1.60	2.54
	½	8.00	1.58	1.01	1.48	2.42	1.53	2.47	1.57	2.52
	⅜	6.10	1.60	1.02	1.46	2.40	1.51	2.45	1.56	2.51
5 × 3	¾	10.88	1.55	0.80	1.31	2.56	1.36	2.61	1.40	2.65
	⅝	9.22	1.57	0.81	1.26	2.52	1.33	2.58	1.39	2.64
	½	7.50	1.59	0.83	1.24	2.49	1.30	2.56	1.35	2.61

Illustrative Prob. 271a. Design a strut of 2 ∟, starred, to carry a load of 32,000# on a 14′-0″ length.

$$\text{Minimum } r = \frac{14 \times 12}{120} = 1.4''$$

r for one angle $= 1.4 \div 2 = 0.7''$

$$\text{Trial area} = \frac{32,000}{10,000} \quad 3.2\square'' \text{ or } 1.6\square'' \text{ per angle.}$$

From tables, 4 × 4 is required (r_{3-3} 0.79″)

$$p = 16,000 - \frac{70 \times 14 \times 12}{0.79 \times 2} \quad 10,330\#/\square'' \text{ allowable}$$

$$A = \frac{32,000}{10,330} = 3.09\square'' \qquad 2 ∟ \; 4 \times 4 \times \tfrac{1}{4} \text{ O.K.}$$

272. Four Angles Starred.

This type of strut is in reality two pairs of double angle struts, and the design may be made in accordance with such restrictions. The radius of gyration is twice that of the double angle strut. The angles should be riveted 2′-0″ o.c. (Art. 206). One use of this type of strut is to reinforce heavy concrete filled columns (Fig. 434 (d)). A strut of this kind is also used where limited space is allotted to the column. Usually equal-legged angles are employed.

Prob. 272a. Design a strut of 4 ∟, starred, 14′-0″ long, to carry a load of 100,000#.

$$\text{Minimum } r = \frac{14 \times 12}{120} = 1.4'' \text{ or } 0.7'' \text{ for each pair of}$$

angles.

$$p \text{ (allowable)} = 16{,}000 - \frac{70 \times 10 \times 12}{1.64} = 10{,}990\#/\square''.$$

$$p \text{ (actual)} = \frac{75{,}000}{9.5} + \frac{10{,}000 \times 5.49 \times 1.99}{2 \times 17.4}$$

$$= 11{,}000\#/\square''. \quad \text{Use } 2 ∟ \ 6 \times 4 \times \tfrac{1}{2}.$$

TABLE 79 — *Continued*

RADII OF GYRATION

Size: Legs	Size: Thick	Area: 2 ∟s	T: 1-1	T: 1-1	3/8″ Gussets: 2-2	3/8″ Gussets: 2-2	1/2″ Gussets: 2-2	1/2″ Gussets: 2-2	3/4″ Gussets: 2-2	3/4″ Gussets: 2-2
5 × 3	3/8	5.72	1.61	0.84	1.20	2.46	1.27	2.52	1.33	2.58
4½ × 3	3/4	10.12	1.39	0.82	1.33	1.84	1.39	2.29	1.45	2.39
	5/8	8.60	1.40	0.83	1.13	2.80	1.37	2.94	1.42	3.01
	1/2	7.00	1.42	0.85	1.29	2.28	1.34	2.28	1.39	2.33
	3/8	5.34	1.44	0.86	1.34	1.88	1.94	1.94	1.85	1.97
4 × 3½	3/4	10.12	1.20	1.01	1.57	1.55	1.48	1.89	1.73	1.92
	5/8	8.60	1.22	1.03	1.61	1.82	1.52	1.90	1.70	1.90
	1/2	7.00	1.23	1.04	1.56	1.81	1.68	1.84	1.69	1.83
	3/8	5.34	1.70	1.07	1.56	1.88	1.60	1.92	1.76	1.98
4 × 3	3/4	9.38	1.87	1.24	1.39	1.05	1.44	2.71	1.49	2.12
	5/8	7.96	1.80	1.22	1.38	1.99	1.41	2.30	1.46	2.08
	1/2	6.50	1.76	1.15	1.33	1.96	1.34	1.96	1.39	2.02
	3/8	4.96	1.26	0.88	1.31	1.94	1.34	1.99	1.40	2.00
	1/4	2.38	1.28	0.89	1.28	1.92	1.33	1.96	1.39	2.02
3½ × 3	3/4	8.62	1.04	0.85	1.42	1.75	1.46	1.79	1.53	1.84
	5/8	7.34	0.87	1.06	1.72	1.41	1.77	1.46	1.82	1.51
	1/2	6.00	0.88	1.07	1.71	1.38	1.75	1.43	1.80	1.48
	3/8	4.60	0.90	1.09	1.67	1.35	1.72	1.40	1.76	1.44
	1/4	3.12	0.91	1.11	1.65	1.34	1.70	1.38	1.74	1.43
3½ × 2½	5/8	6.72	0.69	1.07	1.59	1.16	1.66	1.22	1.71	1.26
	1/2	5.50	0.70	1.09	1.55	1.14	1.65	1.18	1.70	1.24
	3/8	4.22	0.72	1.10	1.52	1.11	1.58	1.16	1.63	1.20
	1/4	2.88	0.74	1.12	1.48	1.08	1.55	1.14	1.49	1.17
3 × 2½	1/2	5.00	0.72	0.91	1.50	1.18	1.55	1.32	1.70	1.23
	3/8	3.84	0.74	0.93	1.48	1.17	1.53	1.21	1.61	1.29
	1/4	2.62	0.75	0.95	1.45	1.13	1.50	1.18	1.58	1.38
3 × 2	1/2	4.50	0.55	0.92	1.56	.094	1.62	0.99	1.49	1.23
	3/8	3.46	0.56	0.94	1.54	0.91	1.59	0.96	1.64	1.01
	1/4	2.38	0.57	0.95	1.52	0.88	1.57	0.93	1.67	0.97
2½ × 2	1/2	4.00	0.56	0.75	1.30	0.99	1.35	1.04	1.41	1.10
	3/8	3.10	0.58	0.77	1.27	0.95	1.34	1.01	1.39	1.06
	1/4	2.12	0.59	0.78	1.25	0.93	1.30	0.98	1.35	1.03

273. Eccentric Loads.

In some cases, beams are framed into struts so that the load is eccentric, as shown in Fig. 431. The design under such circumstances is similar to that discussed in Art. 243. The following problem will indicate the procedure in applying the design principles.

Illustrative Prob. 273a. Design a strut composed of 2 ∟, long legs vertical, 10′-0″ long, for the loading conditions shown in Fig. 431.

$$\text{Minimum } r = \frac{10 \times 12}{120} = 1.0''.$$

Try 2 ∟ 6 × 4 × ½, r (min.) = 1.64″, x = 1.99″ and A = 9.50□″. $I_{1-1} = 2 \times 17.4 = 34.8''^4$.

Prob. 273b. Design a strut composed of 2 ∟, long legs vertical, similar to the conditions in Fig. 431, for a 9′-0″

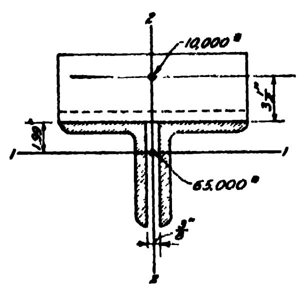


FIG. 431

length, direct load 50,000#, eccentric load 11,000# applied 2″ from face of strut, and angles ½″ back to back.

274. Connections.

The details at the ends of struts and at points where beams frame in are similar to those for larger steel columns. Intermediate beams may be sup- angled strut, the flanges of the beam may be blocked off and the web framed in between the legs as in (*c*) or (*d*). A gusset plate could be used but this increases the eccentricity caused by the beam load.

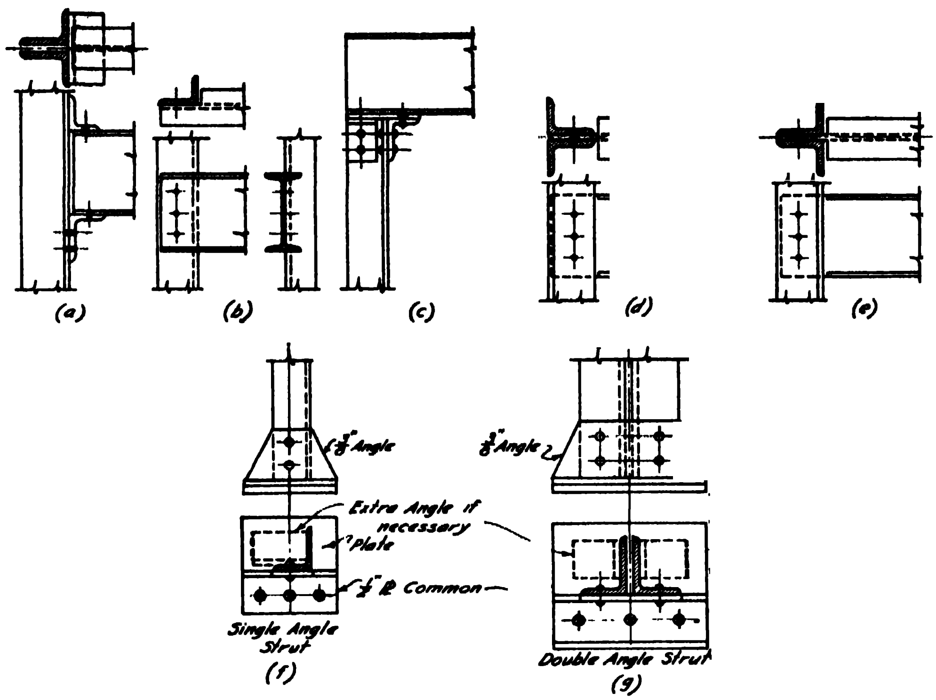


FIG. 432

ported by seat angles (Art. 254) if they frame into the face of a strut as in Fig. 432 (*a*). If an intermediate beam frames into a projecting leg of a single angle strut the web may be attached directly as in (*b*). If there are two projecting legs, as in a double-

When a beam runs over the top of a strut, cap details of angles and a plate may be used, similar to top story columns (Art. 257). Figure 432 (*e*) and (*f*) shows some common details for the bases of struts.

SECTION 23B

CONCRETE-FILLED COLUMNS

275. Uses.

Concrete-filled columns may be used for various types of buildings up to three or four stories high, where relatively light loads are encountered. Advantages are:

(1) relatively small diameters, and hence easily concealed in partitions if required,

(2) quick delivery,

(3) simple erection, and

(4) advantageous use with either steel or wood beams, as shown in Fig. 433.

These columns are considered to be fire-resisting and they have stood up under severe fire and water tests.

276. Types.

Figure 434 shows typical sections which may be used for concrete-filled columns. That in (*a*) is the one used in ordinary cases. When a small amount of additional reinforcement is necessary, a round steel rod may be added, as in (*b*). If several square inches additional are required, a pipe of small diameter may be used, as in (*c*). Another scheme is to add four structural steel angles starred (Art. 272), as illustrated in Fig. 434 (*d*). These offer the best distribution of supplementary metal. The rivets, 6″ o.c., provide good anchorage to the concrete and make a good bond. Sometimes, type (*d*) is designed so that the angles and the concrete

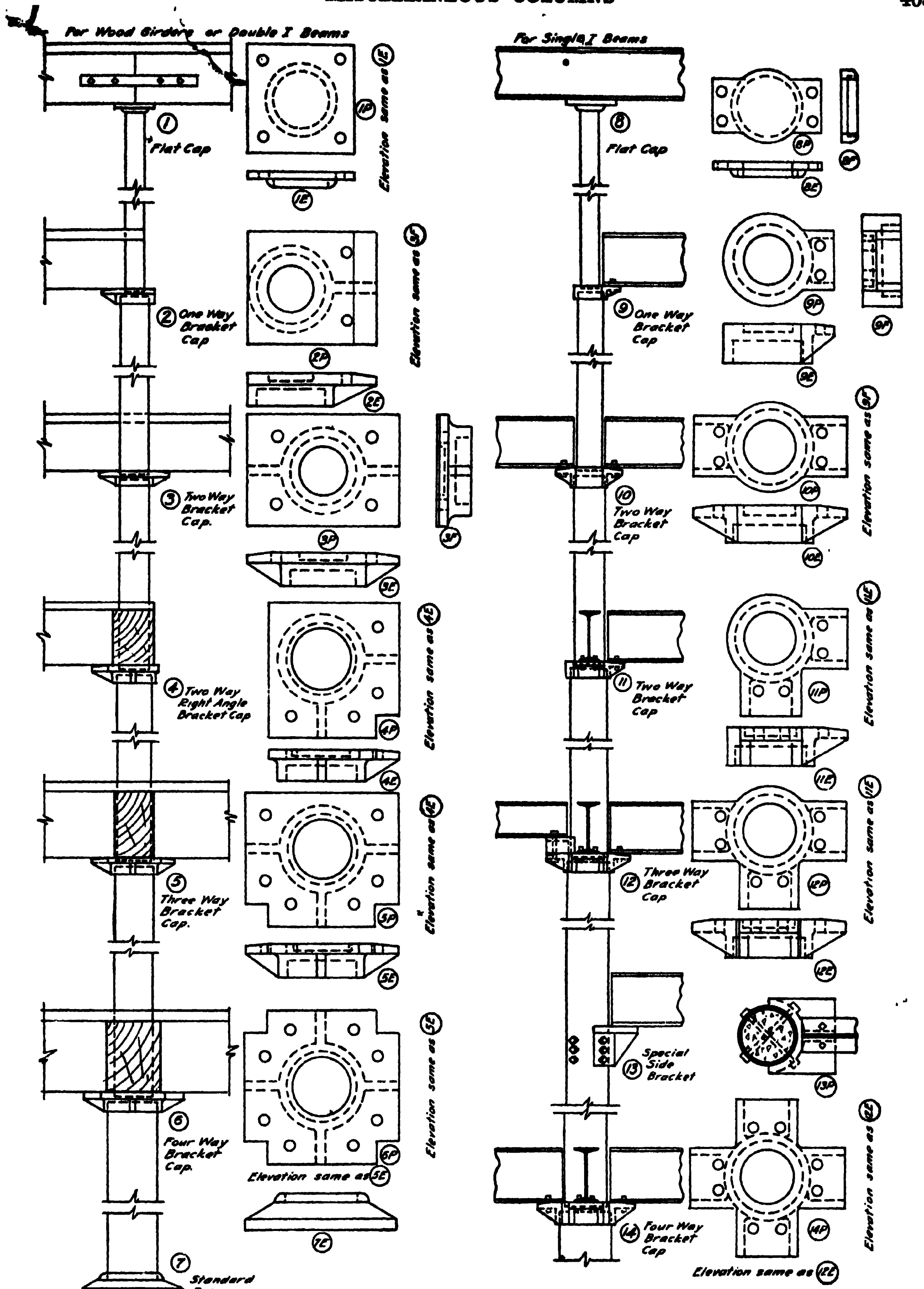


FIG. 433. DETAILS FOR CONCRETE FILLED COLUMNS*

* Courtesy Milford Iron Foundry.

within a circle circumscribed about them will carry the load safely, — neglecting the strength of the pipe and the remainder of the concrete as an extra precaution against fire and abrasion, in order to obtain certain insurance classifications. The supporting power, or at least the hoop strength, of the pipe, however, would not be entirely impaired even in a severe fire.

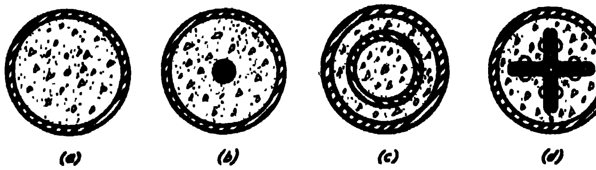


FIG. 434. TYPES OF CONCRETE FILLED COLUMNS

277. Manufacture.

The chief manufacturers who have patents on this type of column are the United States Column Co., Cambridge, Mass., The Milford Iron Foundry, Milford, Mass., the Lally Column Co. of New York City and Chicago, and the Crex Column Co. of Chicago. There are two grades of columns made, — the **light weight** (L.W.), in which single strength standard pipe or casings are used, and the **heavy weight** (H.W.), in which the shell is standard steel pipe.* If the heavy weight columns are not sufficiently strong for a given case, they may be reinforced as suggested in Fig. 434. The pipe stock used for all columns should be new, and the space inside is filled usually with a 1 : 2 : 3 mix of Portland cement, sand, and ½″ crushed stone or gravel, machine mixed. The methods of compacting the concrete vary with the manufacturer, but all aim to secure as dense a filling as possible and to eliminate air voids and honeycomb. One method is described as follows:

"Standing the pipe on end, filling it with concrete from an overhead hopper, and repeatedly raising the pipe and its filling together, bodily, and allowing it to drop on a heavy solid base. In this way the direction of the compacting force is along the long axis of the pipe, and every particle of the concrete filling gets the benefit of every shock, the effect of which is the same as that upon the loose head of a hammer when the butt end of the handle is rapped to tighten the head. That is, the concrete filling acts as a hammer upon itself, and the greater the mass of the filling, the greater the density obtained." †

It is claimed that in this method the coarse aggregate does not tend to segregate to the lower ends of the columns because the crushed rock is purposely kept so small that it lacks sufficient mass to drive down through. Other methods consist of striking blows automatically upon the outside of the pipe at right angles to the axis of the pipe to make the concrete settle and approach its maximum density, or automatically compressing it in a direction parallel to the longitudinal axis with special machinery. It is claimed by some that the method of striking blows is not as efficient as the others because the force of the blow is more or less absorbed by the mass of steel and concrete, especially in large columns.

278. Strength of the Columns.

The ultimate strength of concrete filled columns is such that a factor of safety of 4 or more is obtained when compared with the safe loads as obtained by formula. Progressive tests to ascertain the behavior of this type of column are reported in the catalogue of the Milford Co. ‡ The first object was to establish the relation of the modulus of elasticity of the steel in the pipe to that of the concrete. A typical case is represented by the following:

Figure 435 (*b*) shows the plot of a test on an 8″ × 8″ × 24″ concrete block of a 1 : 2 : 3 mix. The gauge length was 20″. The points on the curve, arbitrarily selected, were the 75,000# and 20,000# loads, or an increment of 55,000#. The corresponding total deformations were 0.0063″ and 0.0008″, or a decrease in length of 0.0055″. Applying the formula for the modulus of elasticity,

$$E_c \quad \frac{P \cdot L}{A \quad e} = \frac{55{,}000 \times 20}{(8 \times 8) \times 0.0055}$$

$$E_c = 3{,}125{,}000\#/\square'' \text{ (with sets deducted).}$$

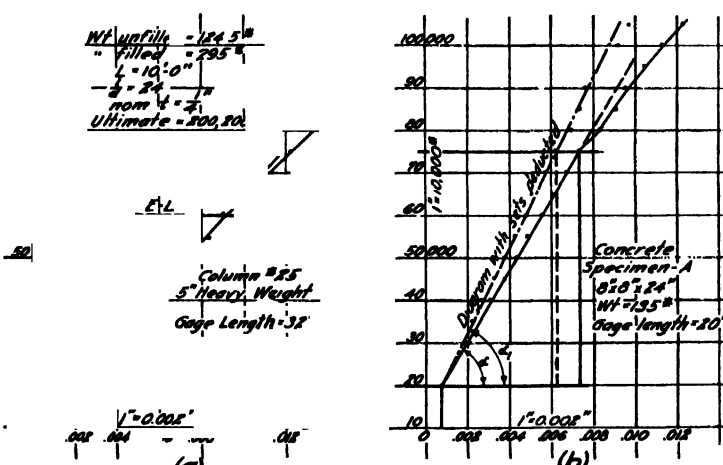


FIG. 435

The corresponding investigation without the sets deducted was

$$E_c' = \frac{55{,}000 \times 20}{(8 \times 8) \times 0.0066} = 2{,}605{,}000\#/\square''.$$

Ratio of

$$\frac{E_s}{E_c} \quad \frac{30{,}000{,}000}{3{,}125{,}000} \quad 9.6.$$

A curve in Fig. 435 (*a*) shows the behavior of a typical column when tested. The following describes the conclusions:

"Reference to the strain readings shows the filled columns to be perfectly elastic at the lower loads, no permanent set

* Extra heavy pipe and also double extra heavy pipe are used for special orders, but this is uncommon, on account of the added expense, special manufacture and consequent delay.

† From the Milford Iron Foundry catalogue.

‡ From tests made at the Massachusetts Institute of Technology, Report of Jan. 26, 1915.

taking place until after the elastic limit is reached. It is evident, therefore, that the steel tube prevents the permanent set in the concrete from taking place below the elastic limit; and the assumption is made that the stress-strain diagram for this concrete, restrained by the pipe, is the same as would be obtained for the unrestrained concrete with sets deducted."*

The curve is composed of two straight lines, — one up to the elastic limit, and the other beyond that point. If the permanent sets were deducted, the second straight line would fall back almost coincident with the first straight line produced. The average elastic limit worked out about 0.3 the ultimate load, so that when the load carried by a column is slightly under the point where permanent set is obtained, it is safe.

The stress at the elastic limit in Fig. 435 was

$$\frac{P}{A} = \frac{75{,}000}{64} = 1170\#/\square''.$$

The stress at the ultimate load on the 8″ × 8″ block was

$$\frac{P}{A} = \frac{259{,}300}{64} = 4050\#/\square''.$$

As a result of the series of tests, the Milford Co. adopted the following formulas:

For ratios of $\frac{l}{d} = 0$ to 15,

$$P = 1100\,(A_c + A_s \times 9.6) \qquad (S\text{–}74)$$

For ratios of $\frac{l}{d} = 15$ to 30,

$$P = (A_c + A_s \times 9.6)\left(1600 - 7\frac{l}{r}\right), \text{ in which} \qquad (S\text{–}75)$$

P = the safe load in lbs.,
A_c = the sectional area of concrete, in □″,
A_s = the sectional area of steel, in □″,
l = the length of the column in ″, and
r = the radius of gyration of the solid section, in ″.

The maximum ratio of $\frac{l}{d}$ should be 30, which corresponds to $\frac{l}{r} = 120$, as r for a solid circular section is $\frac{d}{4}$. The calculations for the column corresponding to the plot in Fig. 435 (*a*) are as follows:

$$A_s = \frac{124.5 \times 144}{490 \times 10} = 3.66\square''$$

$$A_c = 19.64 - 3.66 = 15.98\square''.$$

Concrete equivalent of steel = 3.66 × 9.6 = 35.12□″.
Total equivalent area = 35.12 + 15.98 ≐ 51.10□″.
Stress for concrete at elastic limit,

$$\frac{P}{A} = \frac{60{,}000}{51.10} = 1173\#/\square''.$$

This checks the stress in the unrestrained 8″ × 8″ concrete block (given as 1170#/□″ above) very closely.

* From tests made at the Massachusetts Institute of Technology, Report of Jan. 26, 1915.

Applying the above formula,

$$P = (A_c + A_s \times 9.6)\left(1600 - 7\frac{l}{r}\right)$$

$$P = (15.98 + 3.66 \times 9.6)\left(1600 - \frac{7 \times 10 \times 12}{1.25}\right)$$

$$P = 47{,}500\#\text{ safe load.}$$

The ultimate load was 200,200#, so the factor of safety is

$$\frac{200{,}200}{47{,}500} = 4.22.$$

Table 80 gives safe loads for light weight and heavy weight types of columns, based upon the above formulas. Figure 436 shows typical results of tests of concrete-filled columns.

FIG. 436

The Lally Column Co. gives the following formula for safe loads on their columns:

$$P = A_s\left(13{,}500 - 140\frac{l}{d}\right) + A_c\left(1000 - 11\frac{l}{d}\right).$$

These values agree quite closely with those given in the above table.

Prob. 278a. Check the safe load for a 4″ L.W. column in Table 79 for a 12′-0″ length.

279. Caps.

When beams run over the top of a column, a **flat cap**, similar to that shown in Fig. 437, is generally used, if the loads are relatively light. The width of the plate and the arrangement of the holes may

TABLE 80

SAFE LOADS FOR CONCRETE-FILLED COLUMNS, IN TONS OF 2000 LB.*

Size, column, in.	Weight, column, per ft., lbs.	Nominal thickness of steel, in.	6-ft. length	7-ft. length	8-ft. length	9-ft. length	10-ft. length	11-ft. length	12-ft. length	13-ft. length	14-ft. length	15-ft. length	16-ft. length	17-ft. length	18-ft. length	19-ft. length	20-ft. length
								Light Weight Columns									
3	9	0.109	7	6	5	...	...	...	...	...	...	...	...	...	...	...	...
3½	13	0.120	10	9	8	7	...	...	...	...	...	...	...	...	...	...	..
4	17	0.134	13	13	12	12	11	...	...	...	...	...	...	...	...	...	..
4½	21	0.134	18	17	16	15	14	13	...	...	...	...	...	...	...	...	..
5	26	0.148	24	23	21	20	19	18	17	17	...	..	...	...	...	...	..
6	35	0.165	35	33	32	30	29	28	27	26	25	23	...	...	...	...	..
								Heavy Weight Columns									
2⅞	10	0.203	9	8	7	...	...	...	...	...	...	...	...	...	...	...	...
3½	15	0.216	14	13	12	11	...	...	...	...	...	...	...	...	...	...	...
4	20	0.226	19	18	17	16	15	...	...	...	...	...	...	...	...	...	...
4½	24	0.237	24	23	22	20	18	17	...	...	...	...	...	...	...	...	...
5	29	0.247	28	28	27	25	24	23	22	20	...	...	...	...	...	...	...
5½	36	0.258	34	34	33	32	31	29	28	27	26	...	...	..	...	...	...
6⅝	49	0.280	45	45	45	45	45	43	41	39	37	36	35	33	...	...	...
7⅝	64	0.301	60	60	59	58	58	58	56	54	52	50	49	48	47	46	...
8⅝	81	0.322	75	74	73	72	71	71	71	71	69	66	63	61	60	58	56
9⅝	100	0.342	93	92	91	90	89	88	87	87	87	85	83	81	79	77	75
10¾	123	0.365	112	111	110	109	108	107	106	106	106	106	106	104	101	97	94
11¾	146	0.375	130	129	128	127	126	125	125	125	125	125	125	124	123	120	116
12¾	169	0.375	150	149	148	147	146	145	144	143	142	141	140	139	139	138	136

be made to accommodate wood beams, or double I beams, as in (*a*), or single I beams, as in (*b*).

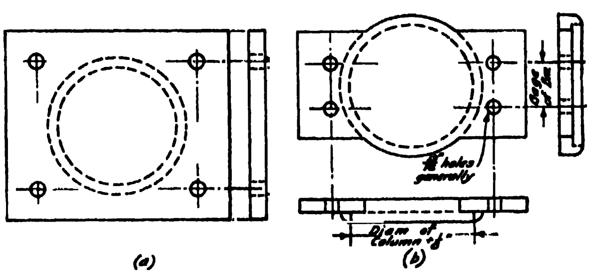


FIG. 437

When a column exists over the one under consideration, **bracket caps** are used, as shown in Fig. 438, (*a*). The wings are purposely made short to keep the eccentricity small. They are beveled $\frac{1}{16}''$ so that the beam reaction will be concentrated as near the column as possible. If the cap is greater than 10″ wide, two brackets are provided under each wing. The cap shown in (*a*) is the Milford type, while that in (*b*) is the Lally standard steel cap. The latter company also has a special screw cap so that the overall length may be altered. This may be an advantage in special erection to eliminate shimming. These caps may be obtained for one-way, two-way opposite, two-way right angle, three-way, and four-way framing, for single wood or double I beams, or for single I beams. These are illustrated in Fig. 433.

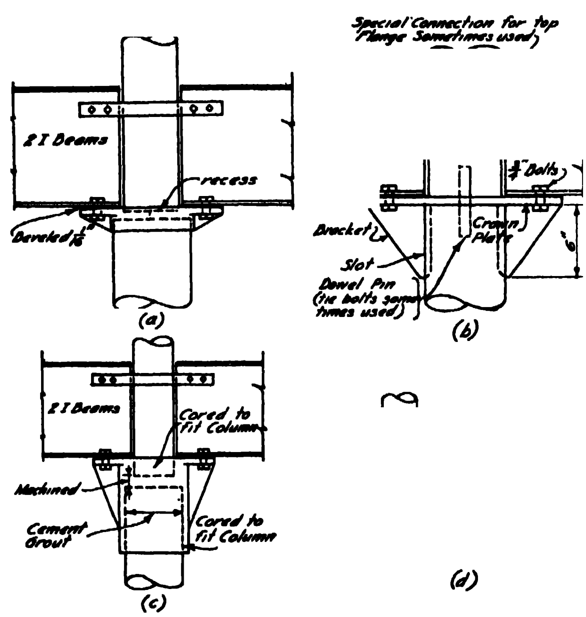


FIG. 438

Figure 438 (*c*) illustrates a Milford cast-iron **socket cap**, which may be used for heavy loads and "through" construction. Figure 438 (*d*) shows a special casting, which may be used to support a

* Milford Columns

beam between floor levels. It is made to match the curvature of the column and is bolted through.

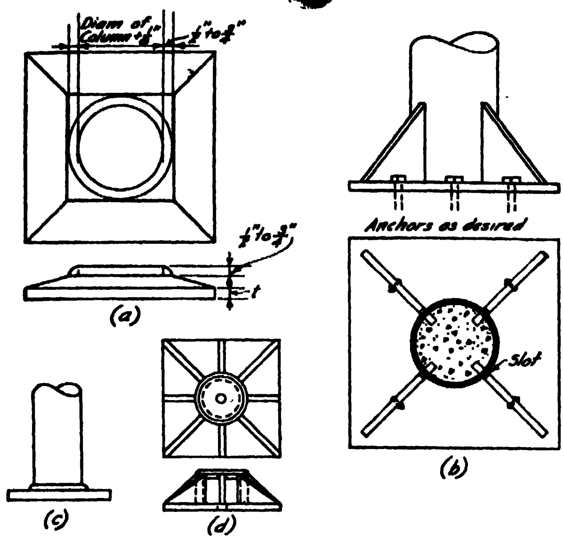


FIG. 439

280. Bases.

Bases for concrete-filled columns, like all others, should have sufficient bearing area to develop the load, and sufficient thickness to distribute the load. Standard bases are usually designed for a bearing pressure of 500#/□", corresponding to bearing on

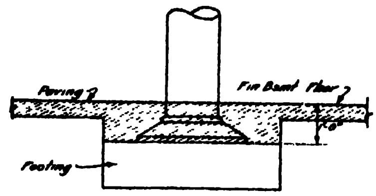


FIG. 440

concrete footings. If some other material is used for a footing, the size of the base should be checked for the case in hand. Figure 439 (a) shows the typical Milford cast-iron base, while (b) illustrates a special Lally built-up base for large-sized columns. That in (c), which is of steel, may be used for small

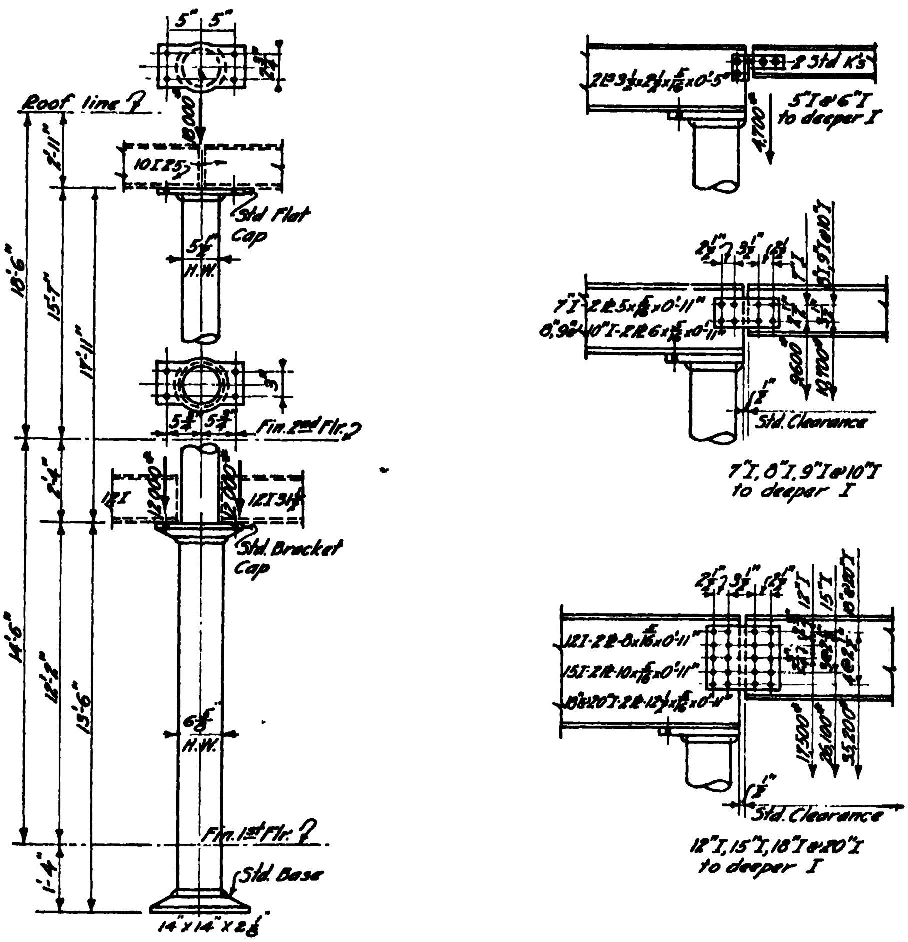


FIG. 441

columns only and for light loads. Figure 439 (*d*) shows a Milford pedestal base for heavy loads, especially when the column is reinforced.

It is usually wise to plan to have the base of the column placed below the basement floor, so that it does not show. Figure 440 shows a method which accomplishes this and still protects the base.

281. Details.

The engineer or a structural fabricating company does not draw complete details for this type of column to furnish to the manufacturer. It is necessary to call for the diameter of the column desired in each case, whether light weight or heavy weight, and the dimensions from base to cap, or from cap to cap. It is also necessary to show the gauges of holes wanted in the caps, and the type of cap and base desired. The manufacturer will supply standard fittings. It is also preferable to show the loads on the columns and the relations to the floor and roof lines. Figure 441 (*a*) shows a typical detail of this kind. Figure 441 (*b*) shows a special method of connecting steel beams over the tops of columns when the depths of the beams are not the same. These are based upon bolted connections in double shear with a unit stress of 6000#/□″.

Section 23c

CAST-IRON COLUMNS

282. Uses, Advantages, and Disadvantages.

SPECIFICATION CLAUSES*

The outside diameter or least side of cast-iron columns shall be not less than 5 inches, nor shall their unsupported length exceed sixty times their least radius of gyration.

The finished thickness of metal in the shaft shall not be less than one-twelfth the outside diameter or the greatest lateral dimension of cross section, nor less than $\frac{3}{4}$ inch. The thickness of metal in flanges, lugs, seats, and brackets shall be not less than 1 inch.

In all cast-iron columns not cast with one open side, at least three holes $\frac{3}{8}$ inch diameter shall be drilled 90 degrees apart near the middle of the shaft for the purpose of measuring the thickness of metal.

Whenever the core of a cast-iron column has shifted more than one-fourth the thickness of the shell, the strength shall be computed assuming the thickness of metal all around equal to the thinnest part, and the column shall be rejected if this computation shows the strength to be less than required by Section 62, paragraph 6.

A cast-iron column shall be rejected whenever blowholes or other imperfections reduce the effective area of the cross-section more than 10 per cent.

The ends of all cast-iron columns shall be planed to a true surface perpendicular to the axis of the column. Successive column lengths shall be bolted together through end flanges with at least four bolts not less than $\frac{3}{4}$ inch in diameter. No shims shall be used between the flanges.

If the core of a cast-iron column below a joint is larger than the core of the column above, the core of the lower column shall be tapered up for a distance of not less than 6 inches, to the size of the core of the column above. In lieu of tapering the core, a steel or cast-iron plate of sufficient thickness may be used between the flanges. The difference between the diameters or sides of any two successive column lengths shall not be greater than 2 inches.

The connection of beams and girders to cast-iron columns shall be effected by means of seats reinforced by brackets of sufficient depth and thickness to support the entire load, and by lugs to which the webs of the beams and girders shall be bolted. The projection of the seat beyond the face of the column shall in general be not greater than 4 inches.

All holes in cast-iron columns shall be drilled. Cored, or cored and reamed holes shall not be permitted. The diameter of holes shall not exceed that of the bolts by more than $\frac{1}{16}$ inch. The distance from the center of a hole to the edge of a flange or lug shall be not less than $1\frac{1}{4}$ inches.

Cast-iron columns shall not be used in any case where the load is sufficiently eccentric to reduce the unit compression to zero in the extreme fibre on one side of the axis of the column.

Cast-iron columns shall not be used in the structural frame of buildings, the height of which is greater than three times their width.

Cast-iron columns shall not be painted or covered until after inspection by the Bureau of Buildings.

* From the Building Code of The National Board of Fire Underwriters, New York City.

Cast-iron columns are seldom used in modern practice on account of their many disadvantages compared with other types of columns the greater demands for safety now made, and the comparatively low cost of steel columns. They are practical only for small buildings of moderate height, say, four stories as a limit, where the fire hazard is reasonably small, although they are more fire-resisting than unprotected structural steel columns. Cast-iron columns are used in mill construction (Chap. 7, Book I) to some extent and are preferred to wood columns by some engineers when the building is equipped with automatic sprinklers. They are still allowed for general use by a few building codes. They have also been widely used for store front construction in the past and probably will continue to serve that end, as they are easily cast in special rectangular shapes which may be ornamented and left as exposed architectural detail. They are fire-resistant to a greater degree than wood, but in case of fire are apt to be

completely shattered by rapid and irregular contraction due to the action of cold water upon the heated metal.

If cast-iron columns are used, a sub-standard fire-protection of metal lath and 1″ cement plaster around them will in many cases prevent failure and prevent collapse when struck by a hose stream and perhaps save a building from ruin.

The connections to cast-iron columns are always field-bolted and consequently do not give a resistance to lateral movement commensurate with that of the field-riveted steel frame. They should be particularly limited to quiescent loads. For this reason cast-iron columns should not be used as a part of skeleton construction. Cast-iron columns take large loads on smaller sectional areas than wood and thus conserve space. The shape of the cross-section and the surface treatments may be varied to suit the demands of the architectural design. The faces, for instance, may be paneled, ornamented or fluted. The loads on cast-iron columns should be nearly concentric and in no case should the development of transverse stresses be allowed to reach any formidable proportions. As cast-iron does not rust as easily as steel, its use in these places and in this exposed way is a distinct advantage.

There are some objections to the use of cast-iron columns and one of the most important is the **uncertain strength** of the material. While the compressive strength is relatively quite high, cast iron is comparatively weak in tension and shear. Average ultimate values are as follows:

Compression	80,000#/□″
Tension	20,000
Shear	25,000

These are affected by the composition, thickness of metal, rate of cooling, and the fineness of the grain. The strengths are often misleading, as internal initial stresses are set up by the cooling, and the amounts of these are indeterminate. The stresses are not as reliable nor as well defined as those for structural steel in spite of the higher compressive strength of the cast iron.

Another objection is the possibility of **imperfections** occurring in the material. The sources of weakness due to unequal cooling are internal strains, rifted seams, and other defects. Others come in the process of casting (Art. 283), such as blowholes, honeycombs, sand spots, and cinder pockets. **Rigid inspection** (Art. 284) **is required** when such flaws are liable to occur. Another objection is the fact that there is **lack of rigidity at the column connections,** because of the bolted connections. **Eccentricity of cores in circular sections** is often a bad feature. This may be caused by a displaced core, either sagging by its own weight or floating because of the buoyancy of the molten metal, and producing unequal thickness around the perimeter.

283. Manufacture.

The manufacture of cast-iron columns is the typical method of making castings, using a sand mold with a core, as illustrated in Fig. 442. It is usually accomplished by casting on the side (flat) or on the end (standing). The first is more commonly used, while the second is preferable but more expensive and requires a deep pit. Many practical details of manufacture must be considered, and only very careful foundry work will produce good results.

In the side method of casting, the buoyancy of the molten metal tends to produce " floating cores " so that the column may be thicker on the under side than on the top, especially near the middle of the length. Hence it is very necessary to hold the core rigidly in place by mechanical means. As the molten metal is forced to rise, it tends to carry some of the " foundry dirt " and air ahead of it. This air may cause " blowholes " or " honeycombs " on the top side, and the foundry dirt may cause " sand spots " or " cinder pockets." The first tendency is overcome by introducing small " vent holes " to relieve the gases at several points, and the second is abated by forcing wire rods through the mold at intervals.

The end method is used to eliminate the tendency toward floating cores, but is satisfactory only for short columns, as the pressure due to a large head of molten metal becomes considerable for the mold to resist. The metal is admitted at the bottom of the mold in this process, and it forces the air and loose dirt out through the top ahead of it, so that a smaller number of the defects enumerated above is probable. In either method of manufacture, care must be used to keep as even a rate of cooling as possible. Unequal cooling is affected by the manner and speed of pouring, and the condition of the mold as to dampness and amounts of covering. Unequal covering causes unequal radiation and results in unequal cooling and all its incident dangers.

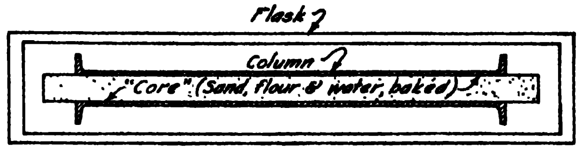


FIG. 442

284. Inspection.

When cast-iron columns are used, it becomes the duty of the structural engineer to give them careful inspection before erection. Defects on the surfaces may be detected by careful scrutiny. The surfaces should be smooth and clean, and the edges sharp. An uneven surface indicates that unequal shrinkage, in cooling, took place, probably caused by an iron which was not uniform in its ingredients. The end view of the section should be fine-grained, with a uniform bluish-gray color, and show a good metallic luster.

Defects inside the surfaces may be discovered by tapping the column with a light hammer. If the sound of the hammer is dull, it may indicate air bubbles, sand holes, soft spots, or honeycomb. Tapping will also indicate the degree of hardness of the metal. Slight indentations are passable, but if small fragments fly off, or there is not the slightest indentation whatsoever, the iron is probably too hard and brittle. Columns are sometimes tested for thickness, blow holes and core floating by drilling two or three $\frac{1}{4}''\phi$ test holes into the shell at random points and inspecting the hole. If the variation in thickness is greater than $\frac{1}{4}''$, the column should be rejected. This process may expose a negligible flaw in an otherwise sound column or, conversely, it may fail to disclose objectionable features in one which is weak. **The only safe precaution when cast-iron columns are employed is to use a large factor of safety.**

285. Sections Used.

Figure 443 shows the usual types of cross-sections of cast-iron columns employed. The circular section is the most commonly used, as it offers a good distribution of metal, — developing a large radius of gyration for the given sectional area. Connections to this type are not as easily made as in some others. The section in (*b*), which may be either square or rectangular, provides nearly as good a distribution of metal as (*a*) and better opportunities for beam connections. The corners are apt to crack when cooled unless great care is taken. The H section, (*c*), has a less economical distribution of metal than the others, but has several advantages. No core is necessary to cast it, it is more easily built into masonry and is thus better protected from fire, and all surfaces are

open to inspection and painting. Other sections have been used in the past such as ribbed-angle shapes, star sections, and so on, but these are now obsolete. Channel sections are occasionally used in store front construction.

The shells are usually made from ¾″ to 2½″ thick and should not exceed 16″ in the largest sectional dimension nor be less than 5″ in the least dimension. The thickness of the shell should not be made to vary by offsets. Sharp corners should be avoided to prevent cracks. Outside corners are rounded and inside corners are strengthened by fillets of ¼″ or ⅜″ radius. It is considered poor design to make one part or section appreciably thicker than some adjoining part or section, as the thinner parts cool quicker and cause a tendency to break away from the thicker parts, inducing cracks.

A variation of this formula is used in the Philadelphia code namely,

$$p = \frac{11{,}670}{1 + \dfrac{l^2}{400\, d^2}} \qquad (S\text{–}76)$$

with a maximum ratio of l to d (where d is the outside diameter of the column in inches) of 20.*

Straight-line formulas are more commonly used in practice because of their simplicity and they are accurate enough for ordinary circumstances. The following formulas represent typical cases:

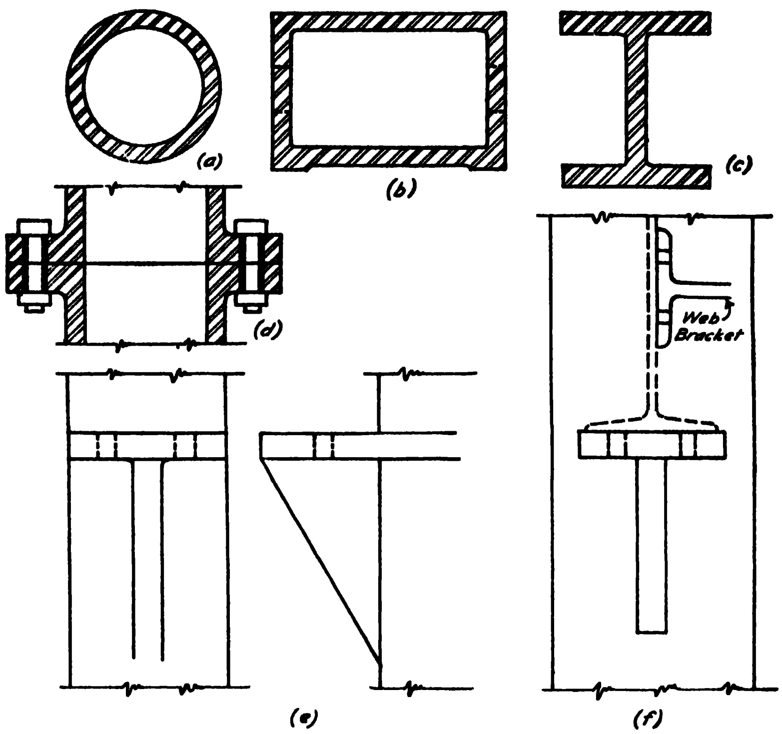


FIG. 443. CAST-IRON COLUMNS

(a), (b) and (c) types
(d) splice
(e) bracket for light beams
(f) bracket for heavy beams

286. General Design.

Because of the many possibilities of flaws and defects in manufacture and the irregular behavior of stresses, cast-iron columns must be designed with a large factor of safety. A value of 8 is common, and some ordinances require 10. Probably the most accurate of the formulas for the design of cast-iron columns is the Rankine or Gordon type. For fixed ends, this is

$$p = \frac{S}{1 + \dfrac{l^2}{5000\, r^2}}, \text{ in which}$$

p = the maximum allowable compressive stress, in #/□″,
S = the maximum allowable bearing stress for cast iron, usually taken as 12,000#/□″,
l = the effective length of the column, in inches, and
r = the radius of gyration of the cross-section, in inches.

$$p = 11{,}300 - 30\frac{l}{r} \text{ (Chicago, Seattle).} \qquad (S\text{–}77)$$

$$p = 10{,}000 - 60\frac{l}{r} \text{ (Boston).} \qquad (S\text{–}78)$$

$$p = 9000 - 40\frac{l}{r} \text{ (New York, Syracuse).} \qquad (S\text{–}79)$$

The maximum ratio of l to r is commonly specified as 70. It will be noted that for an average value of $l \div r$ of 40, that the allowable stresses for the last two formulas are practically the same. Formula S–79 is recommended in the absence of other restrictions. The design of cast-iron columns must be done by "cut and try" methods, — assuming a section and testing

* The Rankine formula is modified by replacing r by d, since r for any thickness is proportional to the outside diameter of the column. Also r changes quite slowly in value for moderate ranges of thickness.

it. The sizes should be limited to those discussed in Art. 285. The properties of a hollow round section* which come into use here are:

Area, $A = \dfrac{\pi (D^2 - d^2)}{4}$ sq. ins., and

Radius of gyration, $r = \dfrac{\sqrt{D^2 + d^2}}{4}$ ins., in which

D = the outside diameter of the column, in inches, and
d = the inside diameter, in inches.

Illustrative Prob. 286a. Design a hollow, round, cast-iron column 18′-0″ long to carry a load of 250,000#. Use formula *S*-79.

Minimum radius of gyration $= \dfrac{18 \times 12}{70} = 3.08''$

From tables of radii, minimum diameter = 10″.

Try 10″φ — 1¼″ thick, A = 34.36□″ and r = 3.13″.

$$p = 9000 - \frac{40 \times 18 \times 12}{3.13} = 6240\#/\square''$$

$$A \text{ required} = \frac{250{,}000}{6240} = 40.1\square'' \text{ n.g.}$$

Try 10″φ — 1⅜″ thick, A = 37.26□″, r = 3.09″

$$p = 9000 - \frac{40 \times 18 \times 12}{3.09} = 6200\#/\square''$$

$$A \text{ required} = \frac{250{,}000}{6200} = 40.3\square'' \text{ n.g.}$$

Try 11″φ — 1¼″ thick, A = 38.29″, r = 3.48″

$$p = 9000 - \frac{40 \times 18 \times 12}{3.48} = 6520\#/\square''$$

$$A \text{ required} = \frac{250{,}000}{6520} = 38.3\square''$$

Use 11″φ — 1¼″ thick.
Wt. = 119.7#/ft.

In some cases it may be cheaper to use a larger diameter column and a thinner section if the space allows. Comparative weights are a guide in this work, as well as the diameter of the column section used above. In practical work of this kind, tables are of considerable help. Table 81 gives allowable loads for cast-iron columns.

Prob. 286b. Design a hollow, round, cast-iron column 16′-0″ long to carry a load of 380,000#. Design by use of formula and check result by table.

Prob. 286c. What size of hollow square column may be used for the data of Prob. 286*b*?

287. Splices.

Sections of cast-iron columns are usually joined together by bolting the flanges of each pair of columns together. These flanges are commonly made a part of the column when cast, although separate pieces are used if the overhang is too large or when ornamented caps are to be used. Figure 444 illustrates some of the common types used. The thickness of the projections should not be less than the thickness of the column. The shear at plane *a*–*a* in Fig. 444 (*a*) should be within safe limits. The area of the flange ring is usually safe in bearing and its size is more often governed by the provision for the bolts. The latter is controlled by the outside diameter of the larger column plus the long diameter of the nut, plus ½″ clearance. A minimum projection of 3″ (as governed by hexagonal nuts on ¾″ bolts) is good practice. The minimum connection should be 4 — ¾″φ machine bolts. The holes for them should be drilled to a templet and reamed so that they will line up properly and give a tight fit. The flanges should be spot faced at the holes to allow full bearing for the nuts and they should be carefully machined on the bearing surfaces to give true and even bearing.†

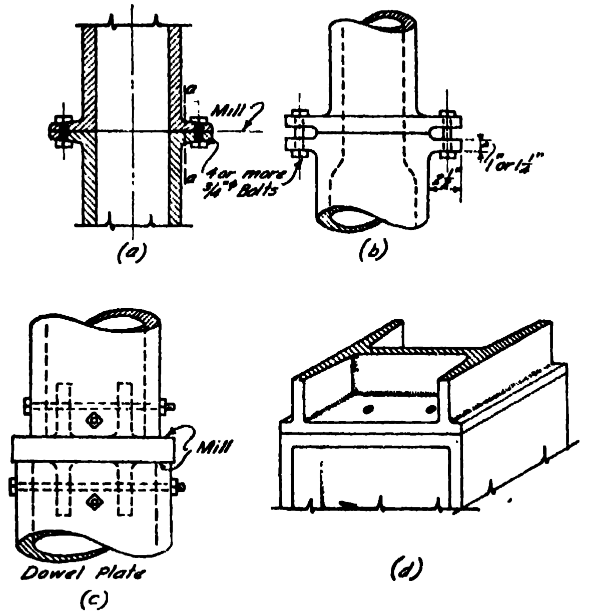


Fig. 444

The columns may change from one diameter to a larger one as illustrated in Fig. 444 (*b*). No sudden offsets should be allowed, as these tend to throw transverse stresses into the column. For large columns, ribs are sometimes cast on to the flanges to stiffen them. H sections may be connected as shown in Fig. 444 (*d*), in which a lip reduces the offset to the upper section, and no flanges are used, thereby keeping the projections small. The bolts are placed within the column, as illustrated. Figure 444 (*c*) shows the use of a dowel plate. This detail is not considered as satisfactory as the use of flanges.

288. Bases.

Bases for cast-iron columns may be cast with the column shaft or separately, depending upon the manufacturer. If separate, the column is less awkward to handle, but the bearing is not quite as reliable. Figure 445 shows common types of bases. Their design is similar to bases of other materials (Art. 261), — providing ample bearing area and sufficient thickness of the projections to resist the induced bending and shear. Type (*a*) may be used for small columns, and it may be square or curved (as shown by the dotted lines), depending upon the area required, and so on. Type (*b*) is used for the heavier columns. The ribs help to distribute the load, stiffen the base plate, and protect the projecting corners. A regular flange (Art. 287) may be used in conjunction with a

* Handbooks give calculated values of areas and radii of gyration for various sections, which will save time. See Carnegie Pocket Companion, Carnegie Steel Co.

† Sometimes the flanges are left rough and sheets (not shims) of paper, lead, or copper are used between them to give an even bearing. This practice is not recommended, and in no case should it be done at the seat of the bottom column.

TABLE 81*

HOLLOW ROUND CAST-IRON COLUMNS

Allowable Loads in Thousands of Pounds

By Formula of New York Building Law, 1916

Weights do not include details

Outer Dia., Inches	Thickness, Inches	Area, Inches²	Weight per Foot, Pounds	Least Radius, Inches	Effective Length of Column in Feet										
					8	10	12	14	16	18	20	22	24	26	28
6	1/2	8.64	27.0	1.95	61	56									
	5/8	10.55	33.0	1.91	74	68									
	3/4	12.37	38.7	1.88	86	80									
	7/8	14.09	44.0	1.84	97	90									
7	5/8	12.52	39.1	2.27	92	86	81								
	3/4	14.73	46.0	2.23	107	101	95								
	7/8	16.84	52.6	2.19	122	115	107								
	1	18.85	58.9	2.15	136	128	119								
8	3/4	17.08	53.4	2.58	128	122	116	109							
	7/8	19.59	61.2	2.54	147	139	132	124							
	1	21.99	68.7	2.50	164	156	147	139							
	1 1/8	24.30	75.9	2.46	181	171	162	152							
9	7/8	22.34	69.8	2.89	171	164	157	149	142						
	1	25.13	78.5	2.85	192	184	175	167	158						
	1 1/8	27.83	87.0	2.81	212	203	193	184	174						
	1 1/4	30.43	95.1	2.78	232	221	211	200	190						
10	1	28.28	88.4	3.20	221	212	204	195	187	178					
	1 1/8	31.37	98.0	3.16	244	235	225	216	206	197					
	1 1/4	34.36	107.4	3.13	267	257	246	235	225	214					
	1 3/8	37.26	116.4	3.09	289	277	266	254	243	231					
11	1 1/8	34.90	109.1	3.51	276	266	257	247	238	228	219				
	1 1/4	38.29	119.7	3.48	302	292	281	271	260	250	239				
	1 3/8	41.58	129.9	3.44	328	316	305	293	281	270	258				
	1 1/2	44.77	139.9	3.40	352	340	327	314	302	289	277				
12	1 1/4	42.22	131.9	3.83	338	327	316	306	295	285	274	264			
	1 3/8	45.90	143.4	3.79	367	355	343	332	320	308	297	285			
	1 1/2	49.48	154.6	3.75	395	382	369	357	344	331	319	306			
	1 5/8	52.97	165.5	3.71	422	408	394	381	367	353	340	326			
13	1 3/8	50.22	156.9	4.14	405	394	382	370	359	347	336	324	312		
	1 1/2	54.19	169.4	4.10	437	424	412	399	386	374	361	348	335		
	1 5/8	58.07	181.5	4.06	468	454	440	427	413	399	385	372	358		
	1 3/4	61.85	193.3	4.03	498	483	468	454	439	424	409	395	380		
14	1 1/2	58.91	184.1	4.45	479	467	454	441	429	416	403	390	378		
	1 5/8	63.18	197.4	4.41	514	500	486	472	459	445	431	417	404		
	1 3/4	67.35	210.5	4.38	547	532	518	503	488	473	459	444	429		
	1 7/8	71.42	223.2	4.34	580	564	548	532	516	501	485	469	453		
15	1 5/8	68.29	213.4	4.76	560	546	532	518	504	491	477	463	449	436	
	1 3/4	72.85	227.6	4.73	597	582	567	552	537	523	508	493	478	463	
	1 7/8	77.31	241.6	4.69	632	617	601	585	569	553	538	522	506	490	
	2	81.68	255.3	4.65	668	651	634	617	600	583	566	550	533	516	
16	1 3/4	78.34	244.8	5.08	646	631	616	601	587	572	557	542	527	513	498
	1 7/8	83.20	260.0	5.04	685	670	654	638	622	606	590	574	559	543	527
	2	87.97	274.9	5.00	724	707	690	673	657	640	623	606	589	572	555
	2 1/8	92.63	289.5	4.96	762	744	726	708	690	672	654	636	619	601	583

* Carnegie Pocket Companion, Carnegie Steel Co. In this pocket book, safe loads for hollow square columns may also be found.

cast-iron pedestal (Art. 284), as shown in Fig. 445 (c), for heavy loads. Sometimes a raised cross, which fits inside the columns and keys them, is cast on the base plates.

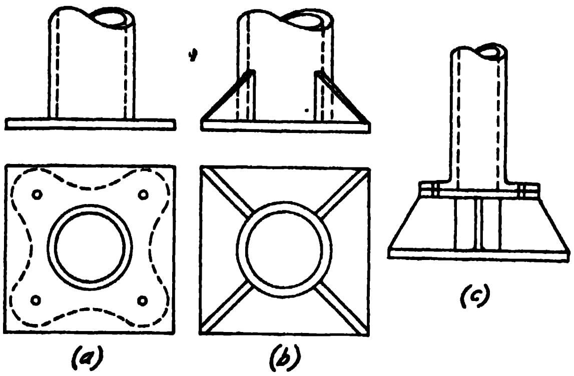


Fig. 445

289. Brackets.

Beams and girders are supported on cast-iron columns by brackets cast integrally with the columns, as illustrated in Fig. 446 (a). The beam is supported sidewise by being bolted to a lug as shown in (b). The lug is also cast with the column. One bracket under the seat is usually sufficient for single I beams and this is placed directly under the web. For box girders, double I beams, and large timbers, there should be two brackets or a double webbed bracket, symmetrically arranged, to eliminate transverse bending as much as possible. A combination lug and separator is used for double beams. The beams are commonly shipped to the job cut to the proper length, but unpunched at the ends. The required holes for the bolts through the lug are drilled in the field to insure a perfect match.

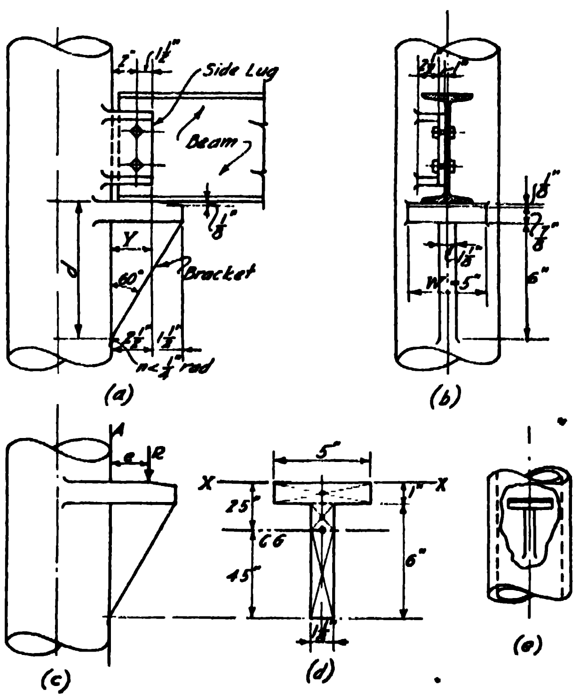


Fig. 446

The bracket should bring a beam reaction as close to the column face as possible in order to reduce the eccentricity on the column. The seat is beveled off $\frac{1}{8}''$ as shown in Fig. 446 (a), to prevent the reaction from being concentrated on the edge of the bracket. The bearing area is usually ample if other requirements are complied with. The width, W, is made large enough to at least match the flange width of the beam or beams supported, and Y is commonly made $2\frac{1}{2}''$. The seat must be square and even in a cross direction so that the reaction will tend to spread out over it and not be concentrated at some "high" point.

Brackets may fail in a number of ways:

(1) By excessive transverse stress due to the bending caused by the load,

(2) By shearing the bracket at the face of the column, as at plane A–A in Fig. 446 (c), or

(3) By tearing out an elliptical hole from the face of the column, as illustrated in (e) (particularly for large-sized columns).

The usual failure is (1), and if a bracket is safe in bending, the shearing and tearing failures are less probable.

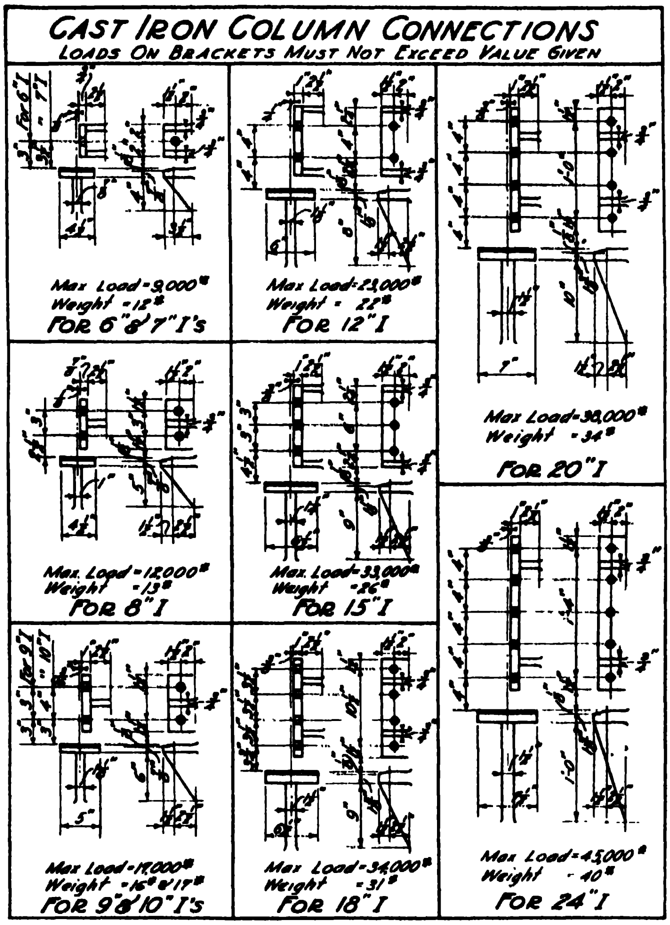


Fig. 447*

The usual practice is to make "standards" for cast-iron brackets, in order to eliminate "cut and try" design as much as possible. Figure 447 shows a set of such standards. The maximum safe loads are determined for each case, checking against the manners of failure discussed above.

* Courtesy of Eastern Bridge and Structural Co., Worcester, Mass.

Illustrative Prob. 289a. Determine the safe load for the bracket shown in Fig. 447 for the 9″ and 10″ I's and in the diagram in Fig. 446 (*d*).

The resisting section is assumed to be that determined by a plane tangent to the face of the column. The first step is to locate the center of gravity of the section, taking moments about plane *X–X* (Fig. 446 (*d*)).

$$y_0 = \frac{5 \times 1 \times \frac{1}{2} + 6 \times 1\frac{1}{8} \times 4}{5 \times 1 + 6 \times 1\frac{1}{8}} = 2.5''.$$

Calculating the moment of inertia,

$$\begin{array}{lcl} I \quad + \quad \Sigma a \cdot d^2 & = & I_0 \\ \frac{5 \times (1)^3}{12} + 5 \times 1 \times (2)^2 & = & 20.42 \\ \frac{1\frac{1}{8} \times (4\frac{1}{2})^3}{12} + 1\frac{1}{8} \times 4\frac{1}{2} \times (2\frac{1}{4})^2 & = & 34.26 \\ \frac{1\frac{1}{8} \times (1\frac{1}{2})^3}{12} + 1\frac{1}{8} \times 1\frac{1}{2} \times (\frac{9}{16})^2 & = & 0.86 \\ \hline \text{Total} & = & 55.54''^4 \end{array}$$

Distance from N.A. to extreme tension fiber = 2.5″ = *c*.

$$\frac{I}{c} = \frac{55.54}{2.5} = 22.2''^3$$

Use F.S. of 10, maximum fiber stress = 2000#/□″. Moment of resistance = $M_r = s\frac{I}{c}$ = 2000 × 22.2 = 44,400″#.

Assume load concentrated at point where beveled portion starts (Fig. 446 (*c*)).

$$M_e = R \cdot e = R \times 2.5$$
$$R \times 2.5 = 44{,}400 \quad \text{or} \quad R = 17{,}800\#$$

Fig. 447 gives 17,000#.

$$\begin{aligned} \text{Shear resistance} &= A \cdot v \\ &= (5 \times 1 + 6 \times 1\tfrac{1}{8})\,2000 = 23{,}500\#. \end{aligned}$$

Prob. 289b. Check the maximum safe load for the bracket for the 18″ I given in Fig. 447. Use same working stresses as in Illustrative Prob. 289*a*.

290. Steel Pipe Columns.

Small columns are occasionally made of steel or wrought-iron pipe, such as for piazza posts and the like. The same limitations should be placed on them as for cast-iron columns, with modified working stresses. Cast-iron caps and bases are generally used. The limiting ratio of thickness to diameter should be $\frac{1}{50}$. A hollow circular section has a maximum radius of gyration for a given amount of metal. Screw caps or bases should not be used on account of the great reduction in sectional area.

PART V

MISCELLANEOUS FRAMING

CHAPTER 24

LINTELS

291. General Considerations.

When openings occur in walls for windows or doorways, some special form of construction is generally necessary, so that the loads above will be transferred to either side of the opening. This is equally true in interior partitions for door openings and interior sash, although the loads may be smaller. Similar circumstances occur over openings for heat registers, ventilation grilles, ducts, and so on.

Provision is usually made in one of two ways,

(1) by the use of arches, or
(2) by employing lintels,*

at the tops of the openings. These are indicated on the floor plan by conventions, as shown in Fig. 448 (*a*). The arch or lintel over the opening immediately below the floor frame is the one indicated. Arches are generally just labeled as such at the openings on structural plans, — the architectural elevations and the $\frac{3}{4}''$ scale details defining them. Lintels are usually defined by typical cross-sections through the openings as shown in Fig. 448 (*b*), (*c*) and (*d*). They are identified at the various openings by marks, such as L1, L2, etc. These marks refer most often to the type of section used. That is, a pair of angles of the same size and relation to the wall section would be given the same "L" number, even if the openings varied slightly. When the structural steel company fabricates the lintels, they assign identifying marks to each, to take care of varying length. Some companies retain the original number and supplement it by a letter; such as L1A, L1B, etc. Sometimes the lintels are further qualified as to their floor location, such as 2L1A, 3L1C, meaning at the second and third floors, respectively. Some structural companies discriminate when the lintel is made up of two or more shipping pieces by the use of a double "L" mark, as LL2A, L3B, the former meaning two separate parts, and the latter, one fabricated unit.

292. Lintels Contrasted with Arches.

An arch is a special arrangement of masonry units, usually built on an arc, so that a balance exists between the thrusts and counterthrusts. The load from an arch causes a component vertical pressure at the supports the same as any beam, but in addi-

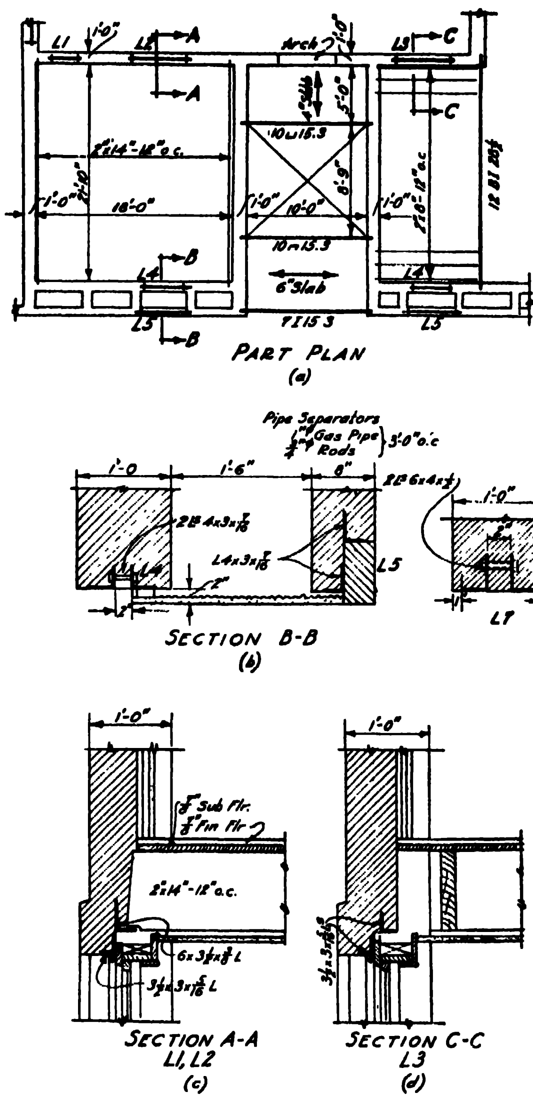


FIG. 448

tion, outward thrusts are created. The supports must be strong enough to resist the joint action of these forces. Figure 449 (*a*) shows the names of the

* Lintels should not be confused with spandrel beams, which are discussed later (see Index). Arches are a part of arch design (see Texts on Masonry Walls).

parts of an arch. The curved line AB is called the intrados and the surface corresponding, the soffit.* The line CD is the extrados and the corresponding surface, the back.* The surface AC is called the face. The stones at the ends are called skewbacks, S, or springers. The stone at the center is called the keystone, K, and the intermediate ones are named voussoirs. The curves are usually segmental, although flat arches may be used.

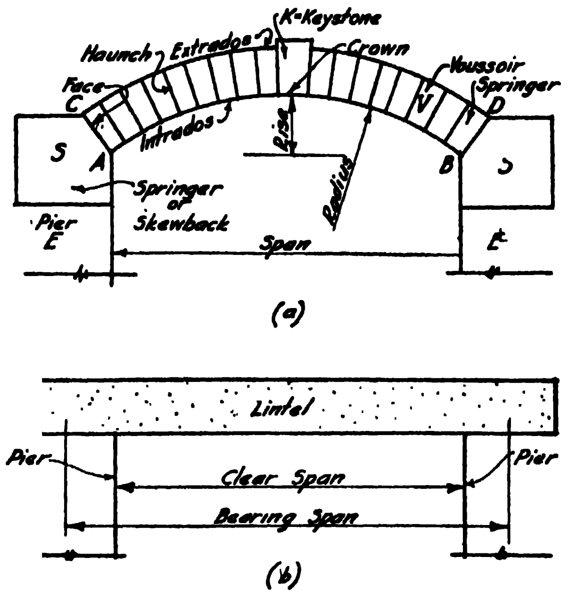


Fig. 449

A lintel is only a special type of beam, and transfers the loads over the opening by virtue of its flexural strength, and causes vertical reactions only (Fig. 449 (b)).

Whether an arch or a lintel is to be used over a particular opening depends largely upon the archi-

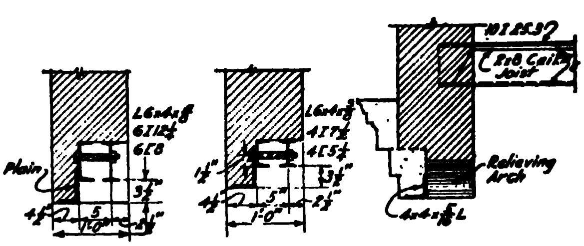


Fig. 450

tectural effect desired (see Vol. I). It is quite common to turn arches in brick work, as pleasing details may be obtained and almost as cheaply as by the use of lintels. Lintels are generally cheaper than arches, particularly where special stone arches are used. Lintels and arches are quite often used in combination as shown in Fig. 450. A relieving arch of brick may be turned back of a lintel carrying the facework, or "face" arches may be relieved by angles in back of them. When segmental arches are used, either curved window heads must be employed, or the trim made to fill in the space between the arch and a horizontal window head.

293. Loads Over Masonry Openings.

Each lintel or arch must be treated as a particular case, as far as the loads on it are concerned. A masonry wall which is solid, possesses considerable arch action in itself, so that the entire mass above the lintel or arch need not always be considered as the effective load. After the mortar has set, the masonry possesses arch action because of the adhesion of the mortar and the natural friction and bonding of the units. However, when the masonry is "green," the arch action has not been fully attained. It also may be lessened by settlement. Hence the amount of load is uncertain and indefinite to some extent. It becomes necessary, therefore, to make assumptions relative to the distribution of the load. A common supposition made relative to the transfer of loads is that they spread along lines making an angle of 60°† with the horizontal, as illustrated in Fig. 451 (a). On this basis, the load on AB, where there is a considerable height of masonry, would be only that contained within the dotted triangle, ABC. If L is the span, the height of the triangle is

$$H = \frac{L}{2} \tan 60° = 0.866\,L.$$

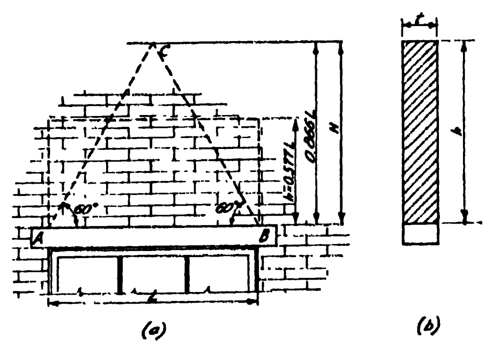


Fig. 451

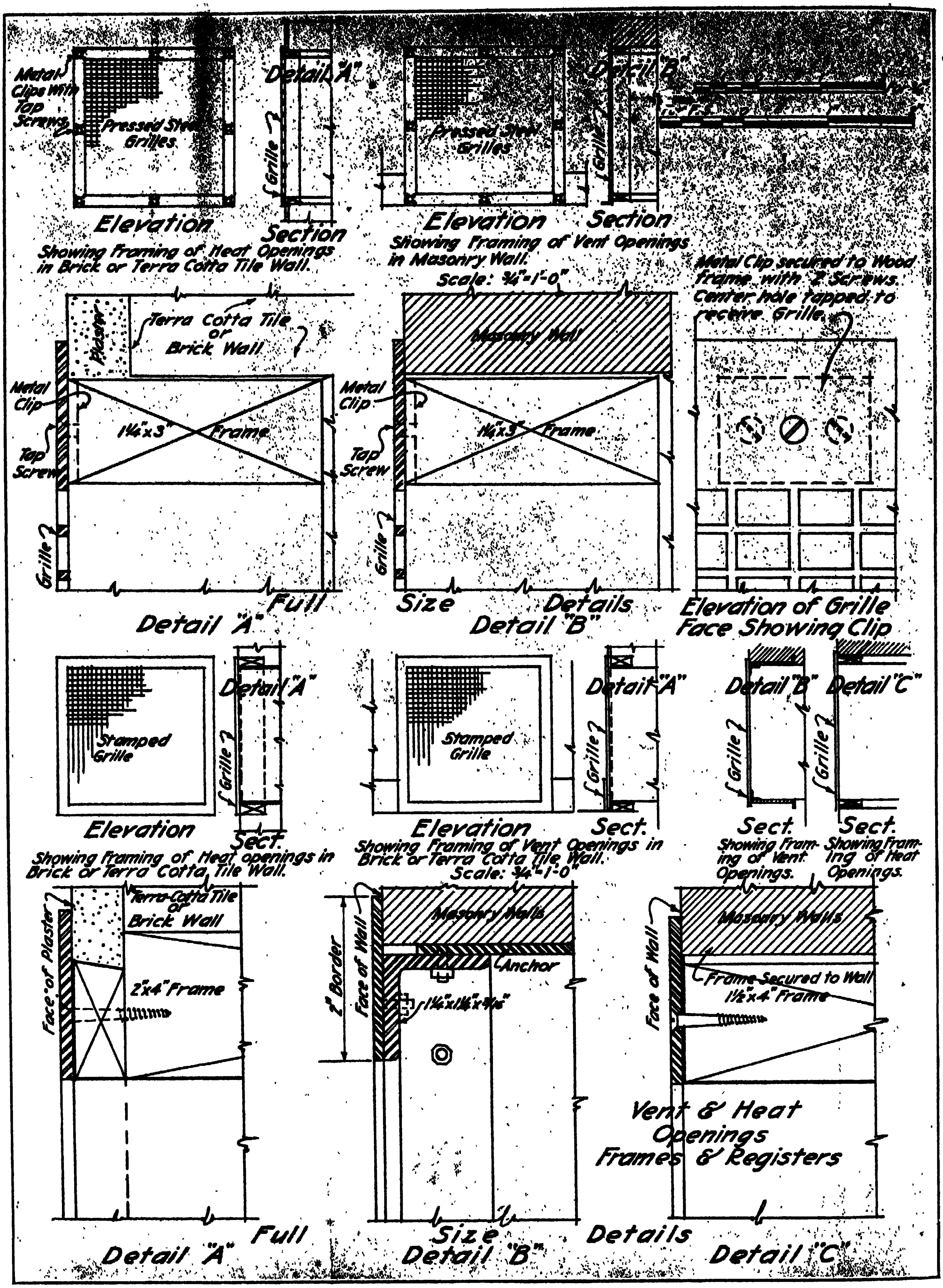
There is a difference of viewpoint in considering this load. The weight of masonry in the triangle ABC is

$$W = \frac{w_s \cdot L \cdot (0.866\,L)}{2} = 0.433\,w_s \cdot L^2, \text{ in which}$$

w_s = the weight per superficial foot of wall of the masonry (= the weight per cu. ft. times the wall thickness).

* This discrimination is not always made and sometimes the lines and surfaces are both called the intrados, or the extrados, as the case may be.

† Some designers assume the angle to be 45°.



Metal Clips With Tap Screws
Pressed Steel Grilles
Detail "A"
Grille
Elevation
Section
Showing Framing of Heat Openings in Brick or Terra Cotta Tile Wall.
Pressed Steel Grilles
Detail "B"
Grille
Elevation
Section
Showing Framing of Vent Openings in Masonry Wall.
Scale: ¾"=1'-0"
Metal Clip secured to Wood frame with 2 Screws. Center hole tapped to receive Grille.
Plaster
Terra Cotta Tile or Brick Wall
Metal Clip
1¼"x3" Frame
Tap Screw
Grille
Masonry Wall
Metal Clip
1¼"x3" Frame
Tap Screw
Grille
Full Size Details
Detail "A"
Detail "B"
Elevation of Grille Face Showing Clip
Stamped Grille
Detail "A"
Grille
Elevation
Sect.
Showing Framing of Heat openings in Brick or Terra Cotta Tile Wall.
Stamped Grille
Detail "A"
Grille
Elevation
Sect.
Showing Framing of Vent Openings in Brick or Terra Cotta Tile Wall.
Scale: ¾"=1'-0"
Detail "B"
Detail "C"
Grille
Sect.
Showing Framing of Vent Openings.
Sect.
Showing Framing of Heat Openings.
Face of Plaster
Terra-Cotta Tile or Brick Wall
2"x4" Frame
2" Border
Face of Wall
Masonry Walls
Anchor
1¼"x1¼"x3/16"
Face of Wall
Masonry Walls
Frame Secured to Wall
1½"x4" Frame
Vent & Heat Openings Frames & Registers
Full Size Details
Detail "A"
Detail "B"
Detail "C"

Fig. 452

This load may be considered as uniformly increasing from the ends to the center. The formula for the bending moment for such loading is

$$M_e = \frac{W \cdot L}{6}, \text{ or substituting,}$$

$$M_e = \frac{0.433\, w_s \cdot L^2\,(L)}{6} \times 12 \text{ in.-lbs., or}$$

$$M_e = 0.866\, w_s \cdot L^3 \text{ in.-lbs.} \qquad (S\text{–}80)$$

Another method is to approximate the average load per foot. The average height, h, of the triangle ABC in Fig. 451 (a) is $\frac{2}{3} \times 0.866\ L$. This figure times the weight per superficial foot gives the load per linear foot on the lintel, or

$$h = \tfrac{2}{3} \times 0.866\,L = 0.577\,L \text{ and}$$
$$w = \text{the load per linear foot}$$
$$= w_s\,(0.577\,L).$$

The bending moment in inch-pounds for a uniformly distributed load is

$$M_e = 1.5\,w \cdot L^2,$$
$$M_e = 1.5\,w_s\,(0.577\,L) \cdot L^2, \text{ or}$$
$$M_e = 0.866\,w_s \cdot L^3 \text{ in.-lbs.}$$

It will be noted that this result is the same as derived above (S–80). Hence either method gives the same results.

The transverse strength of the masonry after the mortar has set tends to relieve the load on a lintel or arch. Referring to Fig. 451 (b), the resisting moment of the section in inch-pounds is

$$M_r = \frac{t \cdot h^2}{6} \times f, \text{ in which}$$

t = the wall thickness in inches,
h = the height of the section under consideration in inches, and
f = the allowable flexural stress of the masonry in #/□″.

The external bending moment may be determined on the basis of restraint, since the wall is continuous by the opening, or

$$M_e = 1.0\,w \cdot L^2$$
$$= \frac{1.0\,(w_0 \cdot t \cdot h)\,L^2}{144}, \text{ in which}$$

w_0 = the weight of the masonry in #/c.f., and
L = the span in feet.

For brickwork, w_0 = 120#/c.f., and f may be limited to 50#/□″. For a 2′-0″ span and a 12″ wall,

$$M_e = 0.866 \times 120 \times (2)^3 = 831''\#$$

$$831 = \frac{f \times 12 \times [0.866(12)]^2}{6} \text{ and } f = 32\#/\square''.$$

Thus it may be seen that **for small openings, say up to 18″, as for heat grilles and so on, no lintels or arches are theoretically necessary. However, small lintels are usually provided for practical reasons.** The flexural strength of the frame in such an opening provides considerable resistance. Figure 452 illustrates typical details for register and vent openings. For larger openings, lintels of minimum sizes should be provided for practical reasons, — to provide against unexpected loads and faults in construction and to give a finish to the opening.

When openings are close together in any story, as illustrated in Fig. 453 (a), there is not much chance left for true arch action, and the load per linear foot

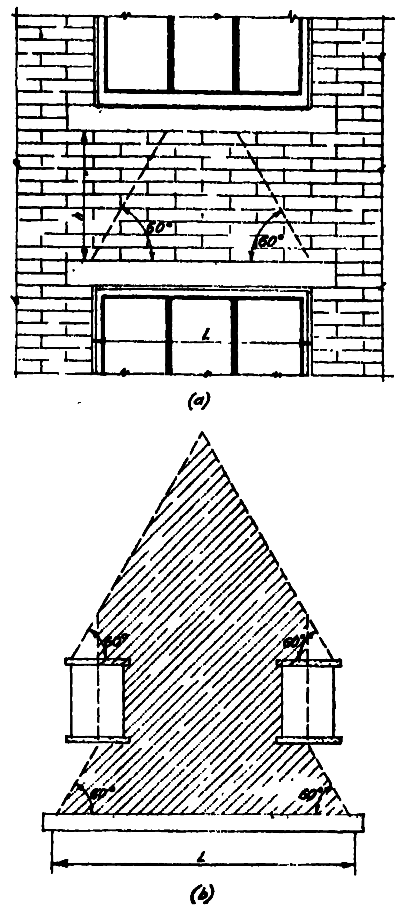
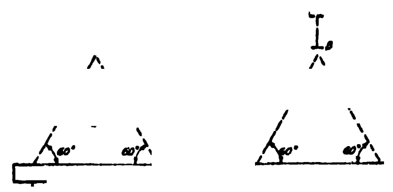


Fig. 453

should be taken as the weight of the masonry, of height, h, plus the weight of the sill and sash above. **When floor loads are carried by the wall, they should also be included.** Figure 453 (b) shows another possible loading condition, and the weight of the wall, cross-hatched, should be included.

When a concentration occurs over an opening, as illustrated in Fig. 454, judgment must be used as to its effect. If it occurs within the 60° triangle, as shown in (a), the safe procedure is to consider it concentrated on the span. If it occurs above the

60° triangle, as in (b), the arch action will tend to throw the load over the opening. To be on the safe side, however, it is wise to add the concentration into the remaining load on the opening and consider the whole uniformly distributed unless the beam is more than L feet above the apex of the imaginary triangle. When a beam practically bears over the



(a) (b)

FIG. 454

opening, it is necessary to use a section which will provide proper connections, although it may have strength greatly in excess of what is theoretically required. As previously stated, **each case must take the surrounding conditions into account.**

294. General Design of Lintels.

The design of lintels involves only the general theory of beams (Part I) with the special considerations characteristic of this kind of a beam. Only the new features connected with such design are discussed in the following articles. The provision of safe flexural resistance is the important part of the design. The deflection should, however, be within safe limits to prevent cracks developing in the wall over the lintel, and to prevent the mortar joints from opening up. The end bearing must be sufficient to prevent overstress. The minimum in all cases should be 4″.* This figure should apply beyond any reveals in the openings. Some engineers prefer a minimum bearing of 6″. There should be a sufficient amount of wall each side of the opening to absorb the thrust caused by the arching effect of the wall over the opening. When arches are used, such expanses of wall must absorb the real thrust.

Many building codes require that **lintels of over 8′-0″ span carrying masonry walls should be fireproofed.** This practice is recommended in the absence of code regulations.

295. Structural Steel Lintels.

When the structural frame of a building is of steel, the lintels are most commonly made of structural steel shapes. The possibilities of varying the section and the fact that they may readily be used in conjunction with other materials lend them to a great variety of uses.

One of the simplest uses of a steel lintel is for the **support of veneered brick work** over openings. Figure 455 illustrates the detail through the head of a typical window, showing an enlarged section of veneer construction on a wood frame. A single angle, usually $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$, or $4 \times 3 \times \frac{3}{8}$, with the long leg vertical is commonly employed. Figure 456 shows a diagram which is helpful in selecting angles for such work. Here the ordinates are the clear spans of openings and the abscissæ are courses of superimposed brick, one brick thick. The curve allows also for the weight of the window and trim above, but for no floor load. The mason contractor is usually asked to build in the steel on a small job of this kind.

Another instance where structural steel angles are similarly used, is to support the brick facing of a concrete lintel. Figure 457 (*a*) and (*b*) shows details of this kind. By using a long vertical leg a reveal in the head section may be provided, as shown in (*b*). It becomes necessary to anchor such angles to the concrete. A common type of anchor is shown in (*a*), namely, $2 \times \frac{1}{4}$ bars $\times$ 0′-10″ bent, placed about 2′-0″ o.c. This detail requires building the angle and its anchors into the form for the concrete lintel. Another type of anchorage is to use $\frac{5}{8}''\phi$ expansion bolts, 2′-0″ o.c., as shown in Fig. 457 (*b*). The angle in this case is attached to the concrete later. A disadvantage is the tendency toward spalling the concrete from the beam bottom and this detail is not advised. One very common method is the use of J-bolts. In this case a bent and threaded $\frac{5}{8}''\ \phi$ or $\frac{3}{4}''\ \phi$ rod is hooked over the reinforcement in the beam, the threaded end is passed through the form and is temporarily fastened with the nut, acting as a tie for the beam side. After stripping, the angle is fastened to the projecting bolts in the usual manner.

A single angle is sometimes used over openings in thin partitions, as shown in Fig. 457 (*c*). For unplastered partitions, the angle is exposed over the opening. This is not particularly desirable. When plastered, difficulty is encountered in furnishing a clinch for the plaster where the upstanding leg occurs. Metal lath should be used as shown. This makes the plaster thin at this point if a normal thickness is to be used.

When terra cotta furring is used on the inside of exterior brick walls, it is necessary to provide a support for this furring over the window heads. The furring is commonly 2″ split tile or 2″ blocks, so that the usual **furring lintels may be $2 \times 2 \times \frac{1}{4}$ angles.** Only a small load is generally present, so that such a size is ample for the usual spans. Figures 458 (*a*) and 466 show typical cases. Such lintels are

* No wall anchors (Art. 16) or bearing plates (Art. 15) are necessary for the usual lintel, although especially heavy lintels should be investigated for bearing. Some specifications call for bearing plates for lintels over 6′-0″ in span.

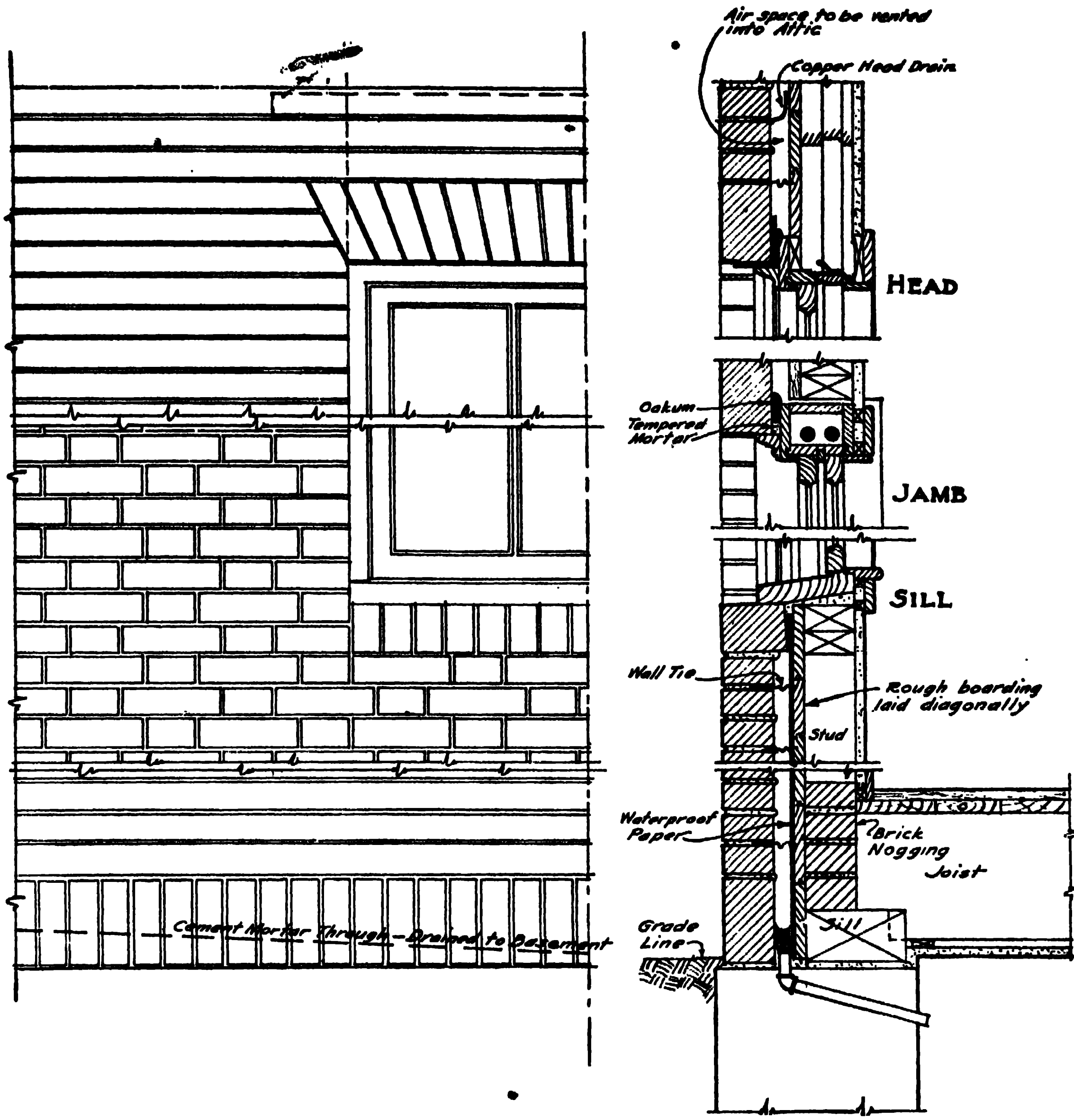


FIG. 455. DETAILS OF BRICK VENEERED WALLS

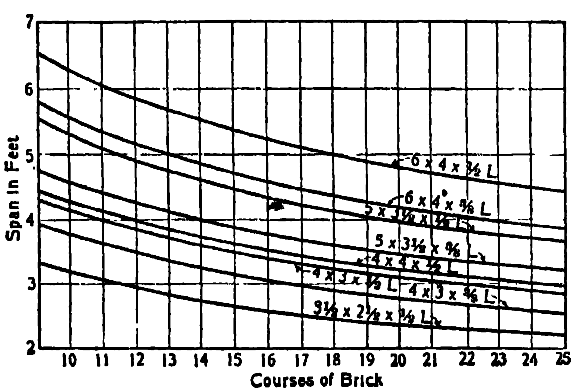


FIG. 456. STRENGTH OF SINGLE ANGLE LINTELS FOR BRICK VENEER WALLS

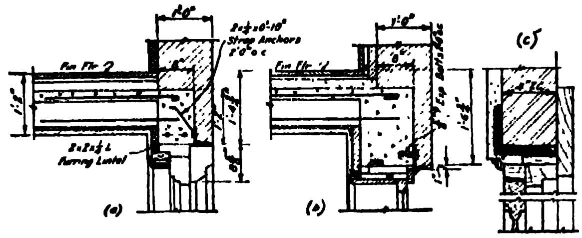


FIG. 457

not shown or called for on structural plans, usually, but are incorporated into the specifications.

When openings occur in masonry walls, a pair of angles is often used to transfer the load over the

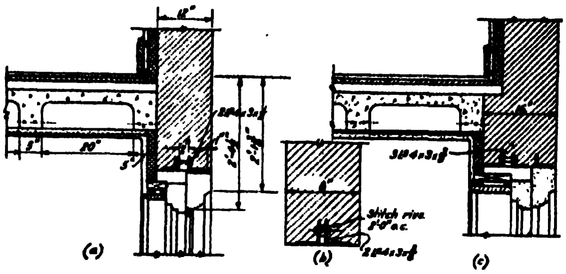


FIG. 458

opening. The outstanding legs should preferably occupy nearly the full thickness of the wall. A figure of ½" to 1" to the face of the wall each side is good practice, as shown in Fig. 458 (*a*). The two angles may be separated by pipe separators (1" ϕ gas pipe and ¾" ϕ bolts, 2'-0" o.c.), as shown in (*a*), or washers may be used. For thinner walls, or where more resistance is required, the angles may be placed back to back, as shown in (*b*). Angles back to back should be riveted together. For thicker walls, more than two angles may be used, as illustrated in (*c*).

For large openings, or where considerable floor load must be carried over openings, angles may not be sufficiently strong, and other combinations of structural steel shapes become necessary. Several schemes may be used, some of which are shown in Fig. 459. That in (*a*) shows a plate and channel. The plate is not counted upon in the lintel's strength but serves as a shelf for the brick. Its thickness should be checked as a cantilever supporting the local load. Walls build up larger than their nominal thickness on account of the joints, and a plate of a width equal to the nominal thickness of the wall may be used. This detail is not preferred by many engineers and some of the others shown are recommended. Figure 460 shows a typical installation of a pair of channels and an angle. When combinations such as shown in Fig. 459 (*c*) or (*d*) are used, the angle is for the purpose of carrying the face course of brick or stone work. Usually it is not counted in the strength of the lintel. The amount that the outstanding leg of the angle is dropped below the channel depends upon the boxing desired in the window head. A common detail when steel sash are to be used is to provide a section such as shown in Fig. 459 (*e*). The shelf angle is riveted to the channel as shown, with separating washers to provide a slot. The steel sash are inserted in this slot and pointed up with grout.

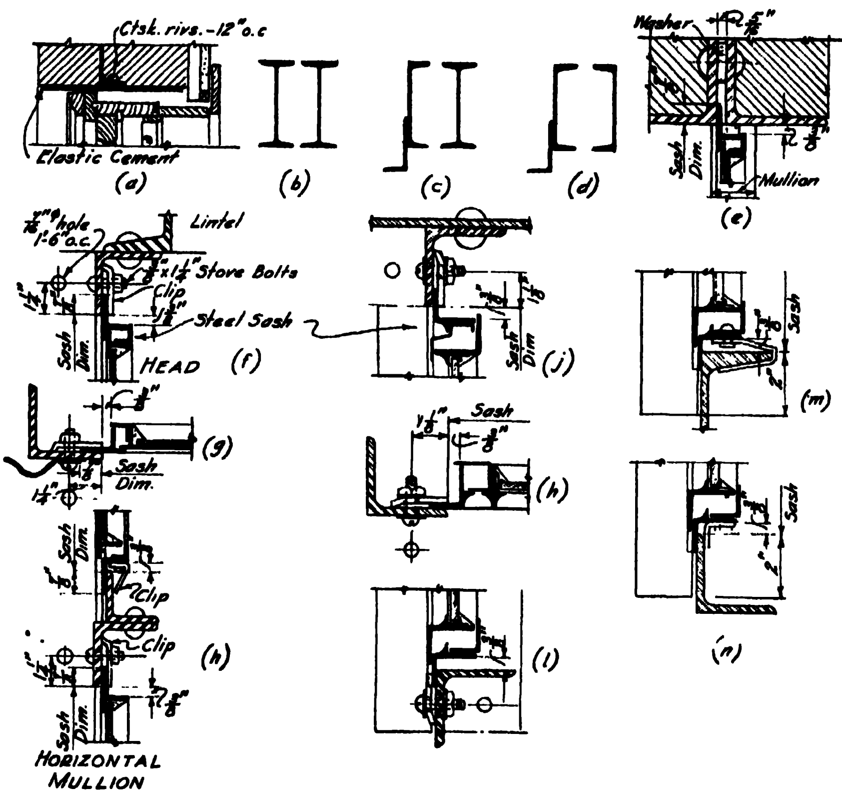


FIG. 459

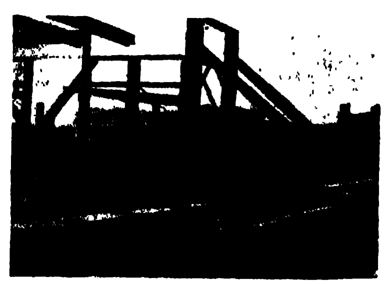


FIG. 460

Figure 459 (*f*) to (*n*) shows several variations for steel sash with different types of girt and lintel sections. Figure 461 shows the use of angle lintels in connection with architectural terra cotta "face" treatments of window heads.

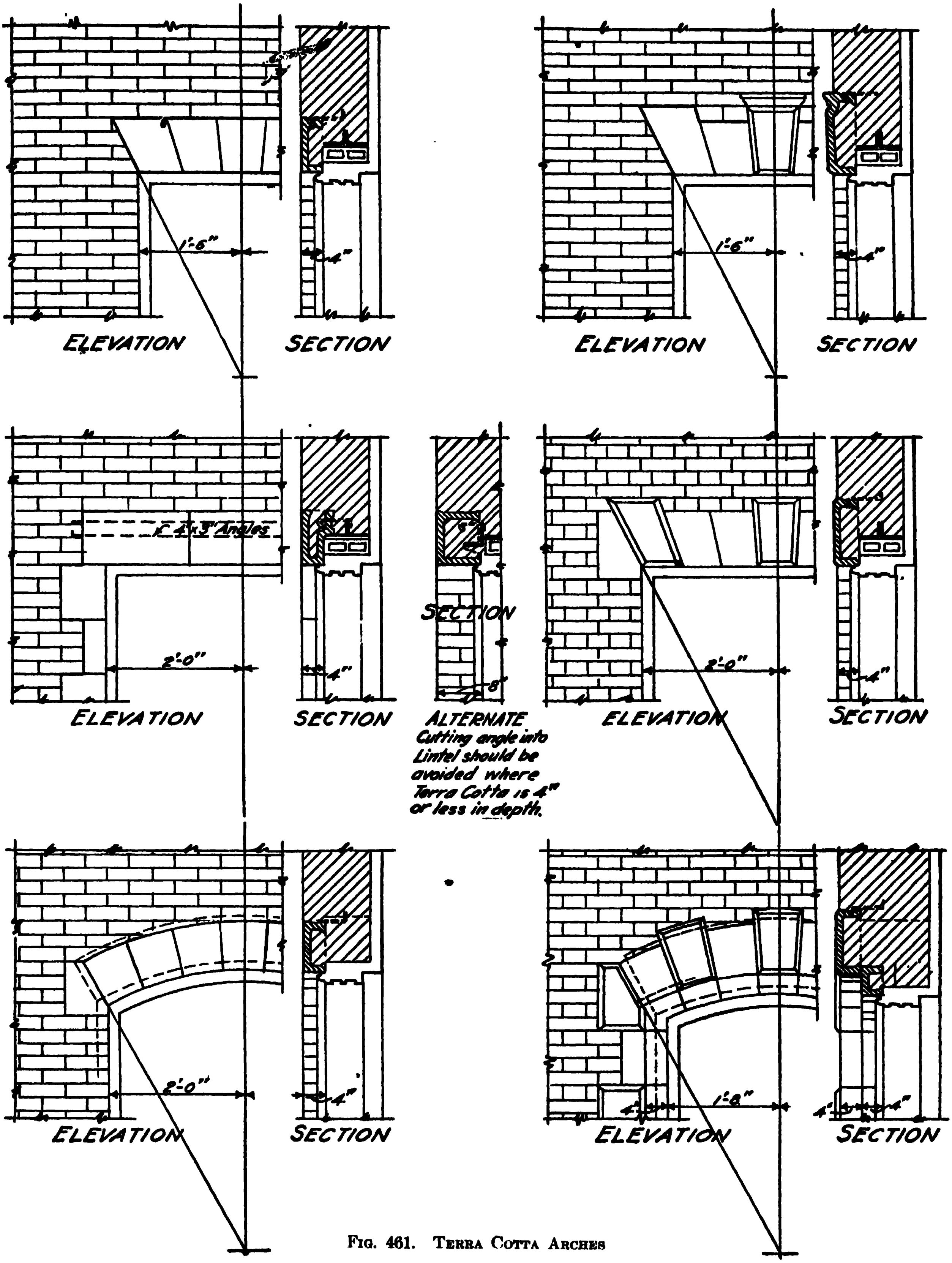


Fig. 461. Terra Cotta Arches

Illustrative Prob. 295a. Design an angle lintel to span a 5′-0″ opening, and carrying a blank brick wall 12″ thick.

Brick work weighs 10# per inch of thickness.
Hence weight per superficial foot = 120# = w_s.
From Art. 293, $M = 0.866\, w_s \cdot L^3$ in.-lbs.
$= 0.866 \times 120 \times (5)^3$
$= 13{,}000''\#$

$$\frac{I}{c} = \frac{13{,}000}{16{,}000} = 0.81''^3$$

From tables, 2 ∟ $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$ are ample.
Space $2\frac{1}{2}''$ back to back, short legs vertical.
Use 1″ pipe separators, space 6″, 2′-0″, 2′0″, and 6″.
Refer to Fig. 458 (*a*) for typical section.

Illustrative Prob. 295b. Design a lintel to span a 7′-6″ opening and to carry 5′-0″ of 12″ brick wall and a floor load of 700# per linear foot of wall. Use 6″ bearing at ends.

Brick work	= 5 × 120 =	600 (Neglect weight of lintel)
Floor	=	700
Total	=	1300#/ft.

$$M = 1.5\, w \cdot L^2 = 1.5 \times 1300 \times 7.5 \times 8.0 = 117{,}000''\#$$

$$\frac{I}{c} = \frac{117{,}000}{16{,}000} = 7.31''^3$$

Use 5 I 10 $\frac{I}{c} = 4.8$

Use 5 ⊏ 6.7 $\frac{I}{c} = 3.0$

Total $= 7.8''^3$

Use $4 \times 3 \times \frac{1}{4}$ shelf angle.
Refer to Fig. 459 (*c*) for typical section.

Prob. 295c. Select a pair of angles to span a 7′-0″ opening to carry a blank 12″ brick wall only.

Prob. 295d. Design a lintel to span an 8′-0″ opening and to carry 6′-0″ of 12″ brick wall and a floor load of 850# per linear foot of wall. Use 6″ bearing at ends.

Prob. 295e. If the lintel in Prob. 295d carried a concentrated load of 12,000# instead of the uniform floor load, what sizes would be required?

296. Special Door Openings.

In planning door openings for elevators, or when other special doors which run on tracks or similar devices occur, the lintels over the openings may require special sections, as illustrated in Fig. 462. The sills also are equipped with light iron when trucking through the doors is anticipated, as shown in (*a*) and (*b*).

297. Beams Carrying Interior Walls or Partitions.

When a beam which is a part of the structural floor framing has to carry an interior wall or partition in addition to the load from the floor, some designers make assumptions as to the weight of the wall material, as it affects the beam.* If a solid wall or partition were built in between two columns, there is a possibility that there would be some arch action, and hence that the load on the beam would be relieved to some extent, providing that the columns were stiff enough to resist any thrust due to such arch action. For such a case, the weight of

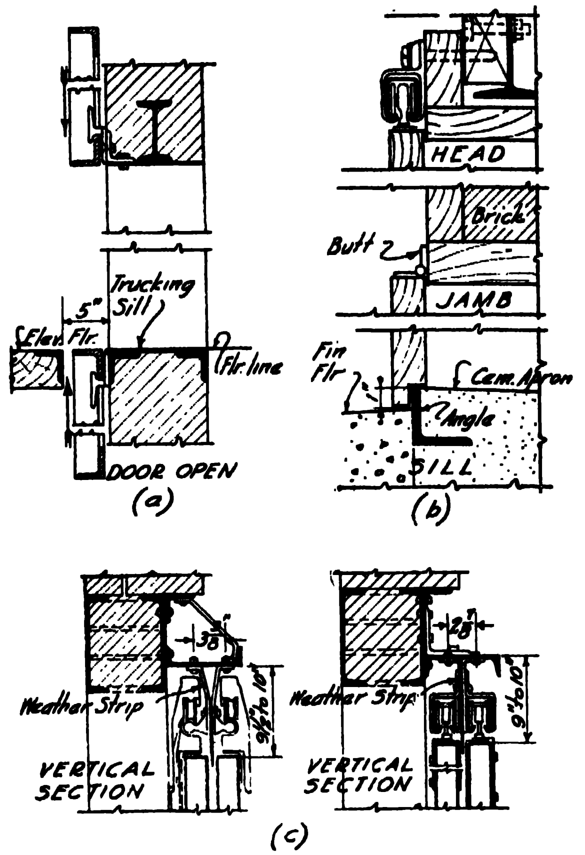


Fig. 462

the wall in Fig. 463 (*a*) is sometimes assumed as that of a triangle of an altitude of $\frac{2}{3}$ *H*. If a door opening occurs in the center, as illustrated in (*b*), some engineers make the same allowance, neglecting the variation in weight for the opening. When a

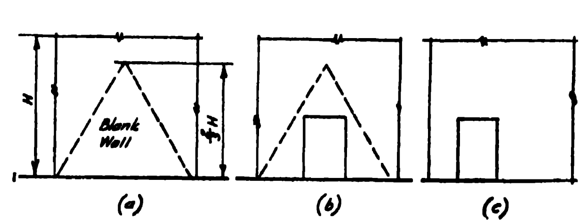


Fig. 463

door occurs to one side, as in Fig. 463 (*c*), or there are two doors, or interior sash, the whole weight is considered, allowing for the difference of weight in the openings. Such conditions as the latter tend to

* Refer to Art. 92 for floor load allowances for random partitions.

destroy any arch action and also to concentrate the weights to a certain extent.

It is not expedient for a structural designer to go too far into detail as to weights of this nature, and it is better to make a liberal allowance and have a reasonably conservative design. The authors recommend that **the full net weight of a wall or partition be used** (weight of wall minus openings*) and that the total be **considered as uniformly distributed.** Neglecting relative positions of any openings, and possible concentrations of loads, will more or less offset each other. Many building code requirements correspond to the above recommendation.

298. Cast-Iron Lintels.

SPECIFICATION CLAUSE†

> Cast-iron lintels shall be not less than $\frac{3}{4}''$ in thickness, and shall not be used for spans exceeding 6 feet.

While cast-iron lintels were employed considerably some years ago, **they are very seldom used in modern practice,** for a number of reasons. The most important of these is that structural steel shapes can be obtained more easily and they are generally more economical and efficient. Other objections are the unreliability of cast iron in bending, the great possibility of imperfections in the metal, and the difficulty of detecting the flaws by inspection. Cast-iron lintels are occasionally used in conjunction with cast-iron columns in store front construction (see Index). These are ornamentally molded for architectural effect.

Figure 464 shows some of the common sections which have been used, with (*f*) indicating a typical elevation. The safe tensile strength of cast iron is customarily specified as 3000#/□″, and the safe compressive strength as 16,000#/□″. It will be noted that the former is only about one-fifth of the latter. Hence it is necessary to have a large part of the section in the bottom flange to supply the required tensile

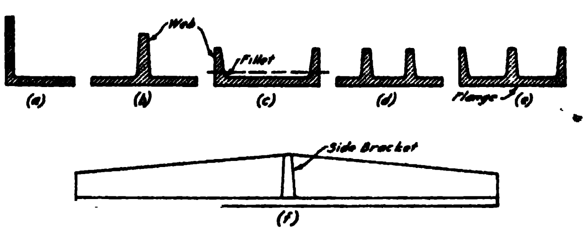


FIG. 464

area. From a theoretical standpoint, the lintel should have equal strengths in tension and compression, but in practice this cannot always be realized. If the thicknesses of the webs and flanges of lintels were materially different, there would be a great tendency toward cracks developing at the juncture of the parts, due to the different rates of cooling after the metal is cast. For this reason, it is good practice to **make the webs and flanges of equal thickness,** although some designers prefer to have the thickness of a web $\frac{1}{8}''$ to $\frac{1}{4}''$ greater than that of the flange.

* The designer should make sure that a blank wall is not the controlling condition for one of the typical beams.

† From the Building Code of the National Board of Fire Underwriters, New York City.

The width of the flange is made to match the thickness of the wall to be supported (or if fireproofed, it is varied accordingly). For widths exceeding 8″, side brackets should be used as in Fig. 464 (*f*), placed at the center of the length of the lintel, to provide lateral stiffness. The depth of the lintel must be sufficient to provide for the bending and to prevent excessive deflection. The webs may be tapered toward the ends, as shown. For a uniform load, this could follow the curve of a parabola, but for practical reasons, it is usually made a straight line. The areas of the webs at the ends must, however, be sufficient to provide ample shearing resistance, not counting upon the flanges beyond the webs. The usual allowable shearing stress is 3000#/□″. Lintels with two or more webs should have a vertical piece at each end to stiffen the webs.

The design of such lintels is usually made commercially by referring to a table of properties of cast-iron lintels of varying sections and of the safe loads that they will carry.‡ Otherwise, the method must be that of "cut and try,"—selecting a given section and determining whether it is safe for a given loading condition or not, and altering the section if necessary. Such design involves the investigation of unsymmetrical sections with different allowable stresses in tension and compression, first locating the neutral axis of the section (through its center of gravity), then determining the section modulus with respect to the two extreme fibres (one in tension and the other in compression), calculating the corresponding moments of resistance, and finally, selecting the controlling value of the latter.§ Usually the resisting moment as governed by the tensile strength controls.

Illustrative Prob. 298a. Determine a cast-iron lintel section to span 7′-0″ and to carry a load of 1600#/ft. Wall 12″ thick. Use section similar to Fig. 464 (*c*). Assume rectangular webs. Assume section 8″ deep and 1″ metal. Calculating the location of the center of gravity by reference to the base,

$$\frac{(8 \times 1) \times 0.5 + 2 \times (7.0 \times 1) \times 4.50}{8 \times 1 + 2\,(7.0 \times 1)} = 3.04'' \text{ up.}$$

$$\begin{aligned}
I \text{ of flange} &= \frac{b \cdot d^3}{12} = \frac{8 \times (1)^3}{12} &&= 0.67''^4\\
A \cdot d^2 &= (8 \times 1) \times (2.54)^2 &&= 51.61\\
I \text{ of 2 webs} &= 2 \times \frac{1 \times (7.0)^3}{12} &&= 57.21\\
A \cdot d^2 \text{ for 2 webs} &= [(1 \times 7.0) \times (1.46)^2]2 &&= \underline{29.01}\\
I & &&= 138.50''^4
\end{aligned}$$

"*c*" distance (tension) = 3.04″
"*c*" distance (compression) = 8.00 − 3.04 = 4.96″

$$\frac{I}{c}\text{ (tension)} = \frac{138.5}{3.04} = 45.5''^3$$

$$\frac{I}{c}\text{ (compression)} = \frac{138.5}{4.96} = 27.9''^3$$

$$M = \frac{s \cdot I}{c} \qquad s \text{ (tension)} = 3000\#/\square''$$

$$s \text{ (compression)} = 16{,}000\#/\square''$$

$$\begin{aligned}
M_r \text{ (compression)} &= 16{,}000 \times 27.9 = 447{,}000''\# \\
M_r \text{ (tension)} &= 3000 \times 42.2 = 126{,}000''\# \text{ (controls)}\\
M &= 1.5\,w \cdot L^2 = 1.5 \times 1600 \times (7)^2 = 118{,}000''\# \text{ (actual)}
\end{aligned}$$

Use Section assumed.

‡ Tables of this kind may be found on page 126 of Hool and Johnson's "Handbook of Building Construction," Vol. I,—McGraw-Hill Book Co., Inc.

§ While cast-iron lintels, as previously stated, are very uncommon in practice, calculations involving their use make excellent theoretical problems for the better understanding of the general theory of flexure.

If the section assumed did not possess a moment of resistance reasonably close to the moment due to the loading, it should be altered. The shear should also be checked up.

$$V = \frac{w \cdot L}{2} = \frac{1600 \times 7}{2} = 5600\#$$

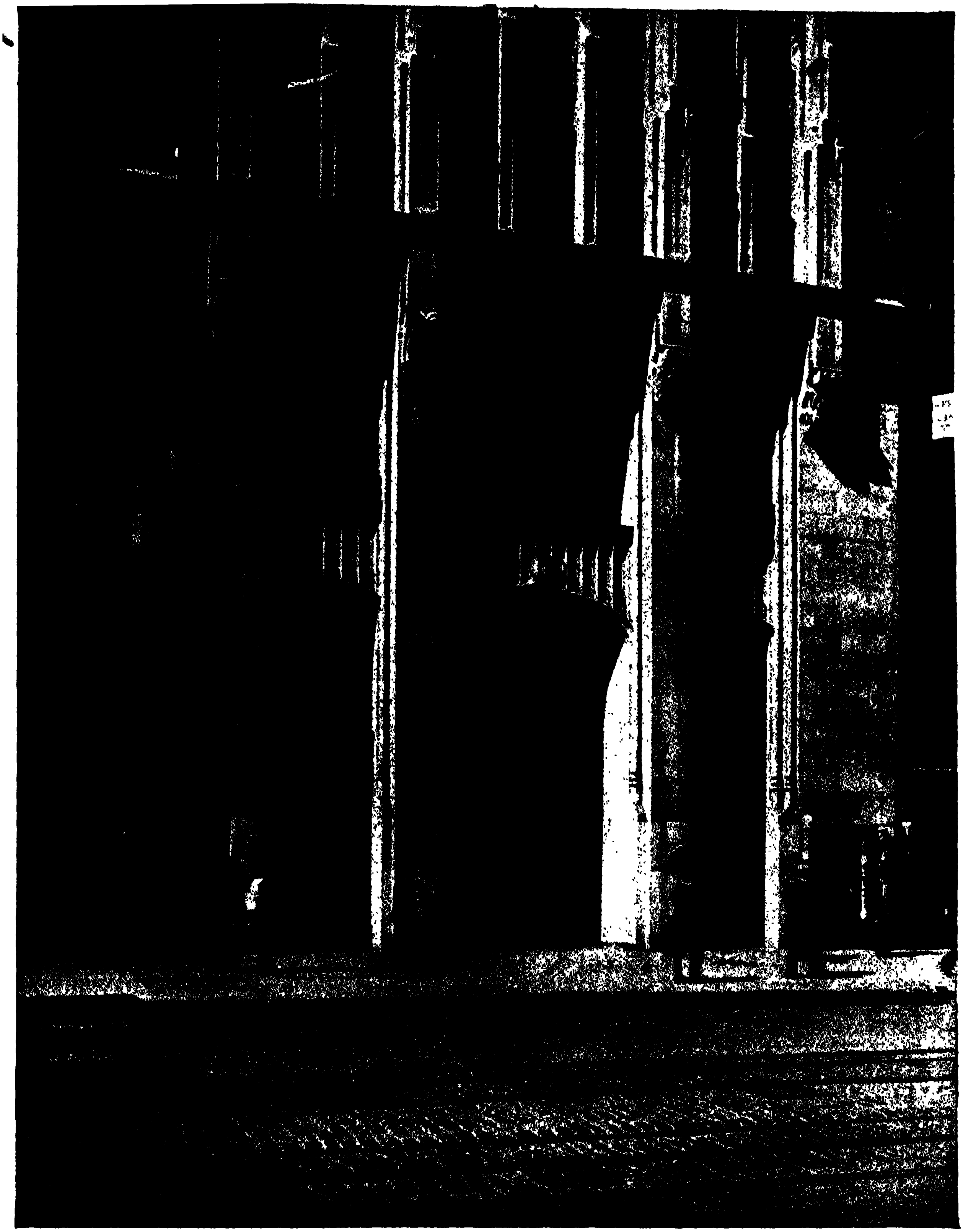
$$\text{Area req'd for shear} = \frac{5600}{3000} = 1.87\square''$$

$$\text{Depth req'd for shear} = \frac{1.87}{1.0} = 1.87''$$

Make 4″ deep for practical reasons.

Use side bracket, as in Fig. 464 (*f*), as lintel is over 8″ wide.

Prob. 298b. Determine a cast-iron lintel section to span 9′-0″ and to carry a load of 900#/ft. Wall 16″ thick. Assume rectangular webs and a section similar to Fig. 464 (*d*). (Hint: Try ¾″ metal and lintel 6″ high.)



'LATE 33 THE OFFICE BUILDING
AIN ENTRANCE, 42ND STREET
IELMLE & CORBETT, ARCHITECTS

CHAPTER 25

EXTERIOR WALL FRAMES*

299. General.

In "skeleton" construction, the exterior walls are curtain walls and are provided principally for the purposes of excluding the elements, and to provide opportunity for architectural treatment. The walls are carried by beams at the floor levels, framing between columns placed near the outside lines of the building. These beams are called **spandrels,** or wall beams. In some cases, they carry a portion of the floor load, as well as the weight of one story of wall construction. This type of framing has become the most common for buildings of any great height, as thick, masonry bearing walls are more expensive and result in slower construction progress for such cases. They have also very poor resistance to earthquake shocks.

In addition to the spandrel framing, special supports become necessary in various types of wall treatments, such as lintels over window and door openings (Chap. 24), parapet walls, cornices (either with or without copings or balustrades), belt courses, window balconies, cantilevered bay windows, porticos and their pediments (Fig. 465 (*b*)) (tympanum framing), main entrance details (Plate 33 and Fig. 465 (*a*)), and so on. In any discussion of limited extent, it would not be possible to take up all such cases, nor all instances of any particular case, but the object of the following articles in this chapter is to point out some of the important features in connection with such work and the design of the structural supports.

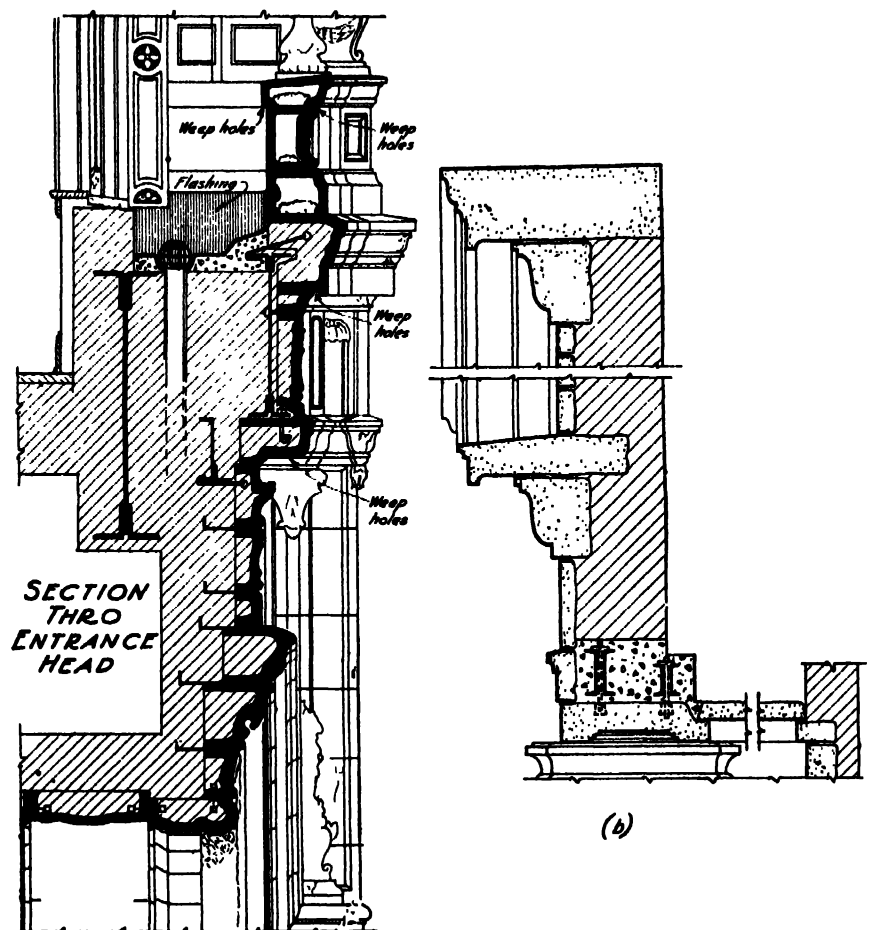


Fig. 465

300. Spandrel Beams.

The design of spandrel beams is not always as simple and straightforward as that of typical floor beams. The first important feature is to plan the

* Lintels and windbracing are often part of this construction and are discussed separately in Chapters 24 and 27 respectively.

arrangement with respect to the cross-section of the wall so that the loads will be carried most efficiently, and so that provision will be made for carrying the facework and for proper anchorage of all important units of the wall construction. Another important feature is to arrange the spandrel beams so that reasonable connections to the supporting columns may be made,* and yet to keep the loads as concentric as possible. The beams should be of simple, rolled sections where possible, and

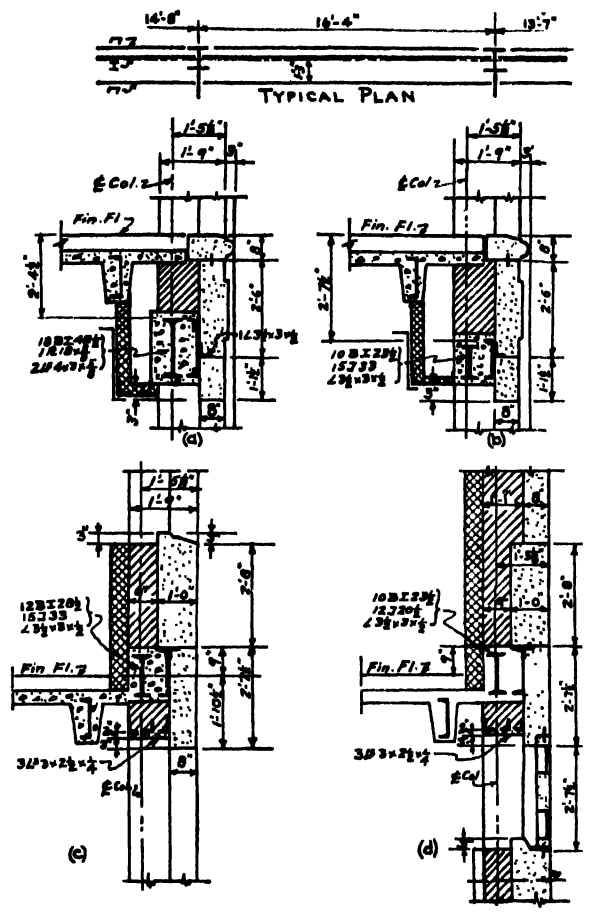


FIG. 466

coping and blocking of beams should be kept to a minimum. One method of helping in the latter situation is to use channels framing between the inside flanges of columns, as shown in Fig. 466, to carry the floor load and any wall furring. This leaves the beams outside, fully available to carry wall load only. If they are thus required to carry less load, rolled sections are more often sufficient.

In calculating the loads on spandrel beams, some care is required to determine the proportionate amount of load that is carried by each member. Where beams of different sizes or depths are used side by side, each particular beam should be designed to carry the load brought upon it. The total load should not be equally divided between the number of beams arbitrarily, unless there is an equal distribution of load. It is important to keep the deflections of the beams side by side as nearly equal as possible. This cannot always be done in an ideal way, because of limiting clearances and space available. In such cases, it is sometimes wise to use separators (see Art. 32) between the beams, to equalize loading conditions as much as possible.

The center lines of the wall columns are usually governed by the fire protection given them. The distance in from the face of the wall to the center lines will be the sum of the protection and one-half the depth of the column section. The latter value must be governed by the lower story columns, which are largest. "Constant dimension" columns (Art. 245) are an aid in this work, and for these, the columns may be kept flush on the outside.

SPECIFICATION CLAUSE†

Protection of Wall Columns All columns which support steel girders carrying exterior walls, and all columns which are built into walls and support floors only, shall be protected against corrosion by a coating of portland cement mortar at least ½ inch thick, and against moisture and fire by a casing of masonry, which shall be not less than 4 inches of brick or 3 inches of concrete on all surfaces; all to be well bonded into the masonry of the enclosing walls.

Note.— Stone work is not reliable protection for steel work against fire. A fire of only moderate intensity is practically sure to cause it to be ruined by spalling, and the metal structural members behind it may thereby be exposed.

It will be seen from the above note that the stone facing is not counted upon as fire protection. Hence the distance from the outside of the wall (when stone facing is used) to the face of the columns must include the thickness of the stone plus at least 4″ of brickwork or 3″ of concrete. In some types of bonded stone work, the thickness of the stones is made 4″ and 8″ alternately, backed up with 8″ and 4″ of brick work, respectively, to the column flanges. Figure 467 shows typical details of bonding ashlar to the backing. The above specification applies to first-class construction. For steel girders and columns which support masonry walls in **second-class construction,‡** other than those facing upon a street, the minimum protection should be 2″, similar to the above specification, or 2″ of metal lath and cement plaster, the latter being applied in two layers with an air space between them.

* Many structural companies provide slotted holes in certain places to take care of the usual variations in setting and plumbing structural work.

† From the "Building Code" of The National Board of Fire Underwriters, New York City.

‡ The protection of metal structural members in non-fireproof buildings is made obligatory only for members supporting walls and main floor sections which in a fire are likely to collapse suddenly with serious danger to firemen and wrecking of the building.

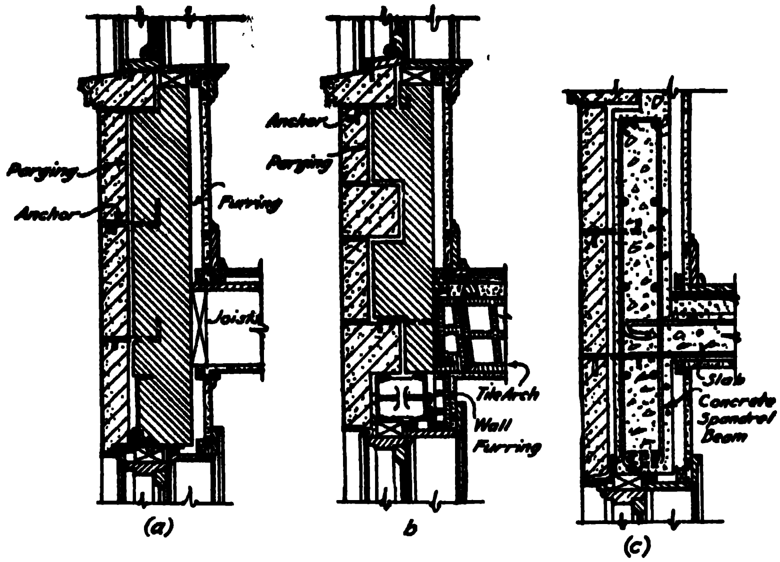


Fig. 467. Bonding Ashlar to Backing

(*a*) in wood frame (*b*) in steel frame (*c*) in concrete frame

Illustrative Prob. 300a. What is the minimum distance from the outside face of a 4″ and 8″ bonded ashlar wall with brick backing to the center-line of the wall columns, if the maximum size of the latter is a 12 BH 118.5?

Ashlar (maximum)	=	8″
Brick backing (minimum)	=	4
Mortar coating	=	¼
½ of 12 BH column	=	6⅜
Total	=	18⅝″, say 1′-6⅝″

The wall girders must also have fire protection. This influences their location.

SPECIFICATION CLAUSE*

Protection of Wall Girders The wall girders shall have a casing of portland cement mortar and the same masonry protection as required for wall columns, all to be securely tied and bonded; but the extreme outer edge of the flanges of beams, or plates or angles connected to the beams, may project within 2 inches of the outside surface of such casing. The inside surfaces of the girders shall be similarly protected by masonry, or if projecting inside the walls, they shall be protected by concrete, terra cotta, or other approved fireproof material not less than 2 inches thick.

Metal fronts on the exterior of buildings over one story high shall be backed up or filled in with masonry not less than 8″ thick.

The simplest spandrel section is of course a single I beam, used for thin, plain brick, terra cotta or concrete-tile curtain walls. Often a Bethlehem section is advantageous on account of its broad flange. For thicker walls, a pair of beams may be used. When a facing material is employed, a section such as shown in Fig. 468 (*a*) may sometimes be adaptable. The angle aids in carrying the facing, and the channel, backed out, allows a surface for connecting the angle. For heavier wall sections, a group of beams such as illustrated in Fig. 468 (*b*) may be used. Not all cases of wall sections can be covered here, but only enough to illustrate the general methods. Considerable ingenuity must be exercised by the engineer in some cases.

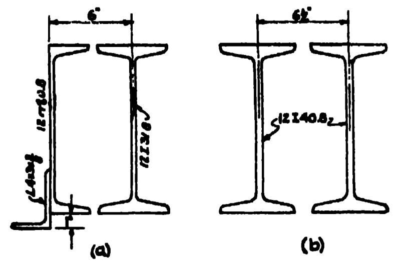


Fig. 468

From a standpoint of connections, the ideal position for a beam which is off the column center-line is so that the inner flange projection may be blocked

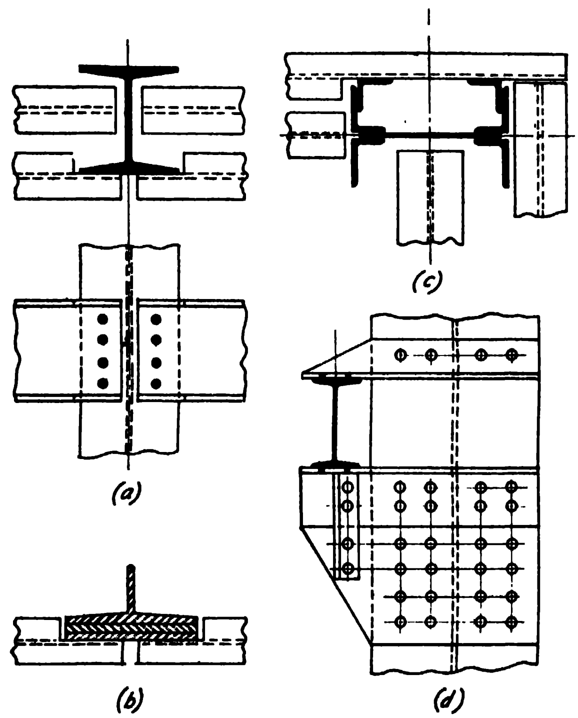


Fig. 469

off and the web connected to the flange of the column, as shown in Fig. 469 (*a*). The center of the beam may be moved out a small distance by the use of packing plates, as shown in (*b*), or "fur-

* "Building Code" of the National Board of Fire Underwriters, New York City.

ring" angles as in (*c*). If the beam must be further out, brackets may be used, as shown in Fig. 469 (*d*). Figure 470 shows another type of spandrel section, which is not afforded as much fire-protection as previously specified, due to less stringent code restrictions.

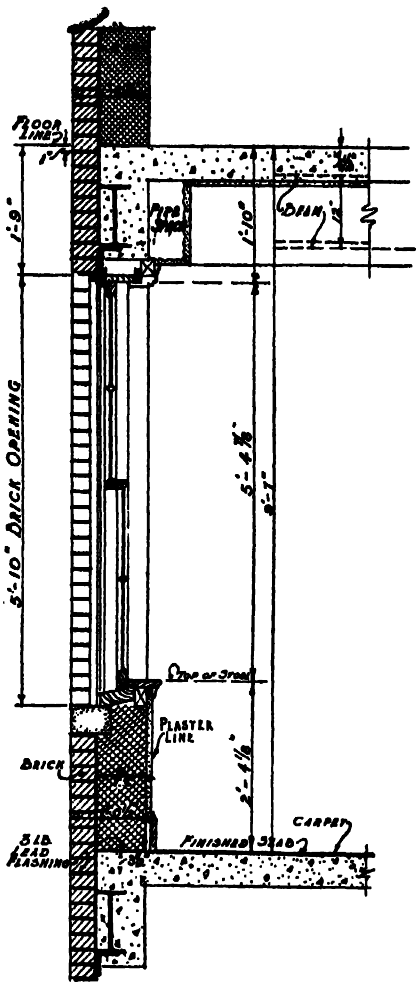


FIG. 470

Illustrative Prob. 300b. Design the typical spandrel girder for the conditions of loading shown in Fig. 371 and summarized in Fig. 471.

Where windows occur

Windows 7.33 × 10	=	73#
Furring and plaster		
(2'-8") + 6" = 3.17 × 10	=	32
Sill $\frac{6 \times 13}{144} \times 150$	=	81
Terra cotta facia	=	80*
12" brick under		
1.0 × (2'-8" + 6'-6" sill)		
2.67 × 1.0 × 120	=	320
Terra cotta around beam, say	=	14
Total	=	600#/ft.

* Ornamental terra cotta averages about 100#/c.f. in weight.

Piers

Brick 1.67 × 12.0 × 120	=	2400
Furring and plaster		
10.0 × 12.0	=	120
Total	=	2520#/ft.

Floor Load

L.L.	=	75
Fin.Flr.		
(⅞" maple)	=	4
Fill	=	24
Random partitions	=	15
Terra cotta arches	=	24
Steel frame	=	5
Plaster	=	10
T.L.	=	167#/□'

Concentration from floor beam $= \frac{5.0 \times 14.0}{2} \times 167 = 5830\#$

Assume girder weighs 50#/ft.
Figure 471 (*b*) shows the loading diagram.
Beam symmetrically loaded.

$R_1 = 5830 + 2520 \times 3.0 + 600 \times 4.5 + 50 \times 7.5 = 16{,}465\#$

Maximum moment occurs at mid-span, *B*.

$M = 16{,}465 \times 7.5 - 5830 \times 2.5 - 2520 \times 3 \times 6.0 - 600 \times 4.5 \times 2.25 - 50 \times 7.5 \times 3.75 = 55{,}810'\#$

$\frac{I}{c} = \frac{55{,}810 \times 12}{16{,}000} = 41.8''^3$ Use 12 I 40.8

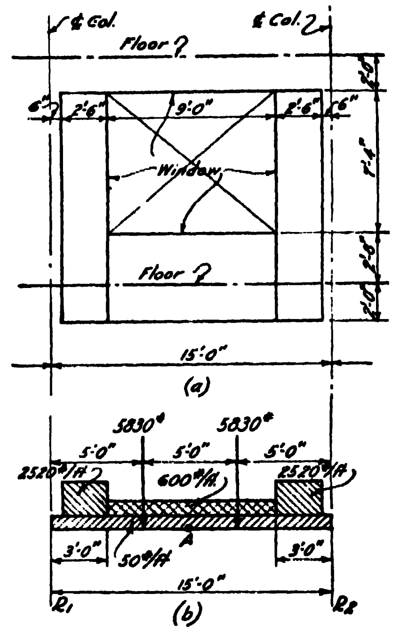


FIG. 471

Prob. 300c. Design the typical spandrel beam for the conditions of loading shown in Fig. 371 and similar to the details in Fig. 471 (*a*).

301. Cornice Supports.

While cornices are used to enhance the beauty and architectural treatment of a building near the

roof line, there is a tendency at times for an architect to make them quite ponderous, in order to carry out the spirit of his design, and to lose sight of the limitations of his materials. Usually the amounts of the projections and the weights can be kept within reasonable limits if study is given to it. An engineer should use his influence with an architect in such designing, as heavy, projecting cornices are a menace during a fire, or in localities subject to earthquakes, and simple, balanced cornices will be much less complicated and expensive to support. Many building codes limit the cornice projection to a definite amount on account of the above reasons.

The important feature in investigating cornice supports is to determine whether the construction is "balanced" or not. This applies in a similar way to belt courses, friezes, and the like. Cornices which are constructed so as to be self-supporting, structural masonry, must be made up of stones which are permanently in stable equilibrium upon the center of gravity of the immediate wall section. Prudence dictates that they should be anchored down in addition, and they should never depend upon superimposed masonry for stability alone. This is illustrated in Fig. 472. Even with anchorage, equilibrium is assured only by verifying that the weight of each portion of a stone inside the vertical line through the center of gravity shall predominate the weight of each portion outside of this line. The weight of the masonry above any point of anchorage should be more than the uplift on the anchorage, as a safeguard. Even in balustrades and copings, one should not rely upon the mortar joints and superimposed weight for stability, but all joints should be provided with bronze dowels and anchors to secure the parts against movement, as shown in Fig. 473. **When a vertical line through the center of gravity of the cornice section falls outside of the immediate wall section, special framing of lookouts and anchors must be used,** such as shown in Fig. 474. Figures 475 and 476 show some alternate details. Various combinations of angles (as outlookers), bolts, pins, rod and plate anchors, pipes, and washers may be used to hold the various stones and to transfer their loads to beams and channels, which in turn are carried by the wall columns. Figure 477 shows several of these combinations.

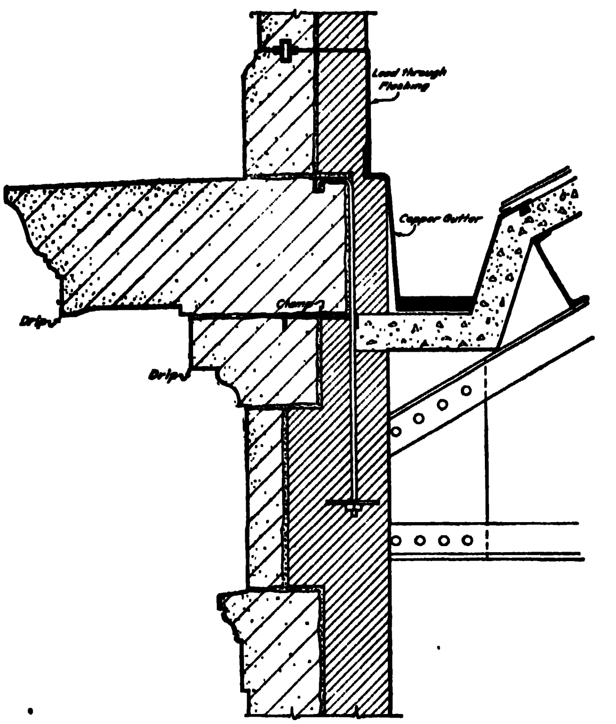


FIG. 472. ANCHORED CORNICE STONES

Illustrative Prob. 301a. Provide means of support for the cornice and the adjoining wall shown in Fig. 478. Columns 19′-0″ o.c. Use outlookers at third points, or 6′-4″ o.c. Assume terra cotta blocks as solid and at 100#/c.f. for the purpose of calculating weights, and to make a conservative allowance for the engaged brick work. For regular brick work, allow 120#/c.f.

In order to estimate the weights, a drawing at a reasonably large scale, similar to Fig. 479, should be made. This is also helpful in arranging the parts.

Load on outside channel. (Blocks scaled.)

Block $A = 0.92 \times 0.4 = 0.37$		
$B = 1.0 \times 0.35 = 0.35$		
$C = 1.7 \times 0.4 = 0.68$		
1.40 c.f. @ 100	=	140#/ft.
Brick work engaged by B and C = 0.5 c.f. @ 120	=	60
Assume $1'$-$0''$ of load from top of cornice	=	68
	T. L. =	268#/ft.

Top of Cornice

L. L.	= 30
Roofing	= 6
2¼″ Cinder Concrete (ave.)	= 20
2″ Book Tile	= 10
Angles	= 2
T.L.	= 68#/□′

$L = 6'$-$4'' = 6.33'$ partially continuous

$M = 1.2 \times 268 \times (6.33)^2 = 12{,}400''$#

$\frac{I}{c} = \frac{12{,}400}{16{,}000} = 0.76''^3$ 4 [5¼ O.K.

Concentration on outlooker = $258 \times 6.33 = 1630$#

Block $D = 1.2 \times 0.7 = 0.84$ c.f. @ $100 = 84$#

Assume this occurs for one-half of the distance

Concentration at first pair of angles

$84 \times 6.33 \div 2$	= 266
Brick work $0.33 \times 1.0 \times 120 = 40 \times 6.33$	= 254
Top of cornice 68×6.33	= 430
Total	= 950#

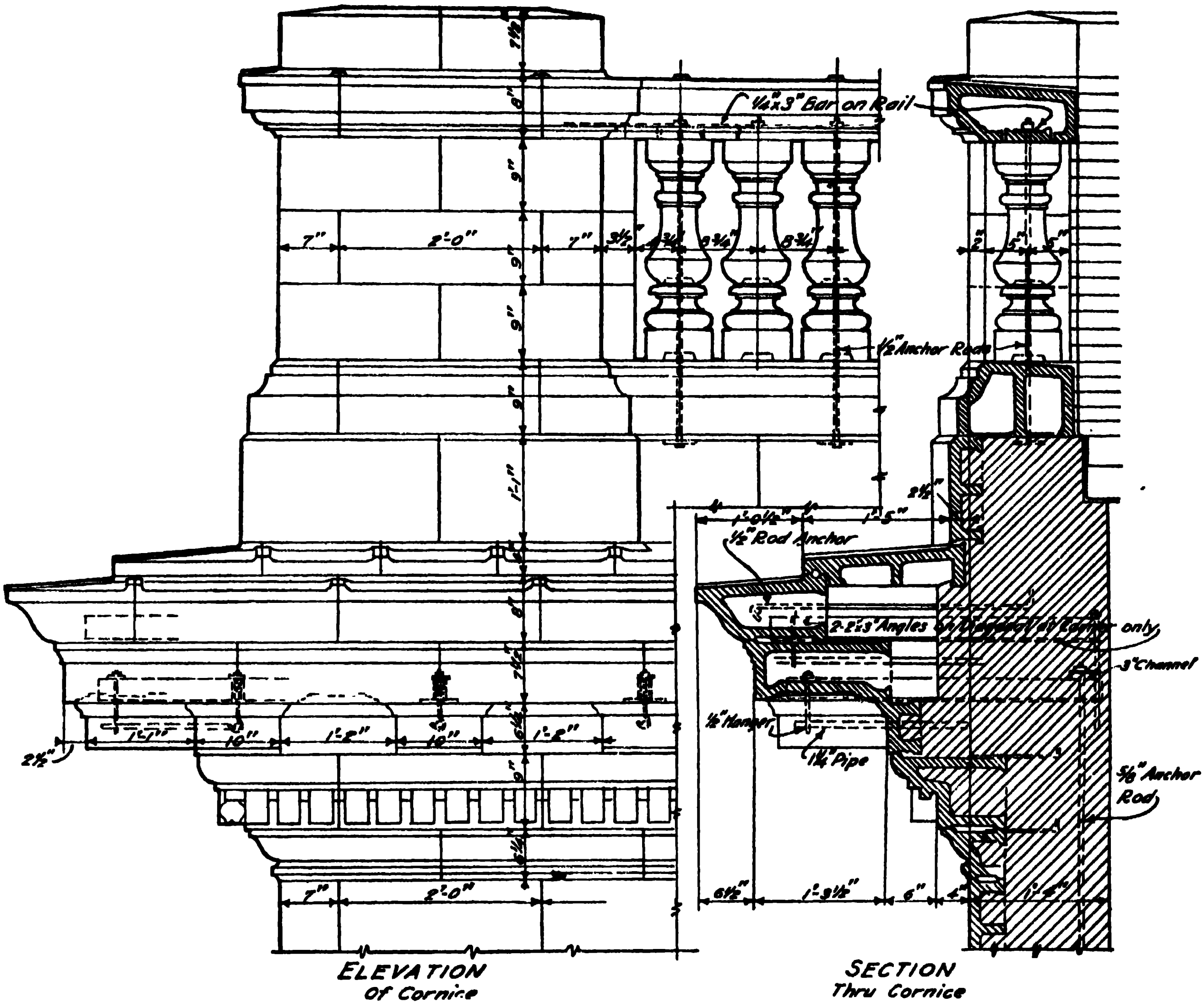


FIG. 473. TERRA COTTA CORNICE AND BALUSTRADE

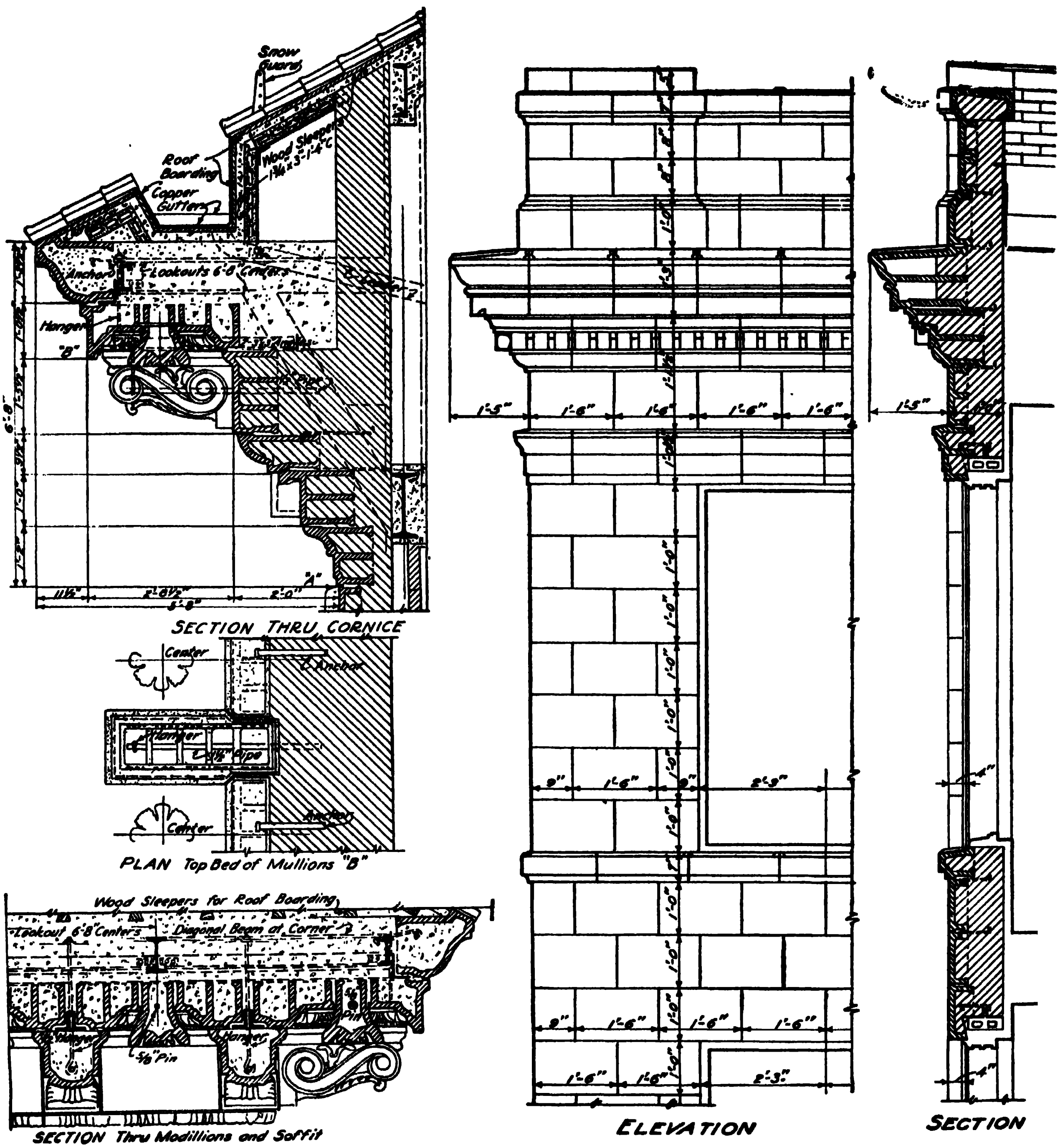


Fig. 474

(a) section of typical terra cotta cornice

(b) terra cotta spandrel in brick wall

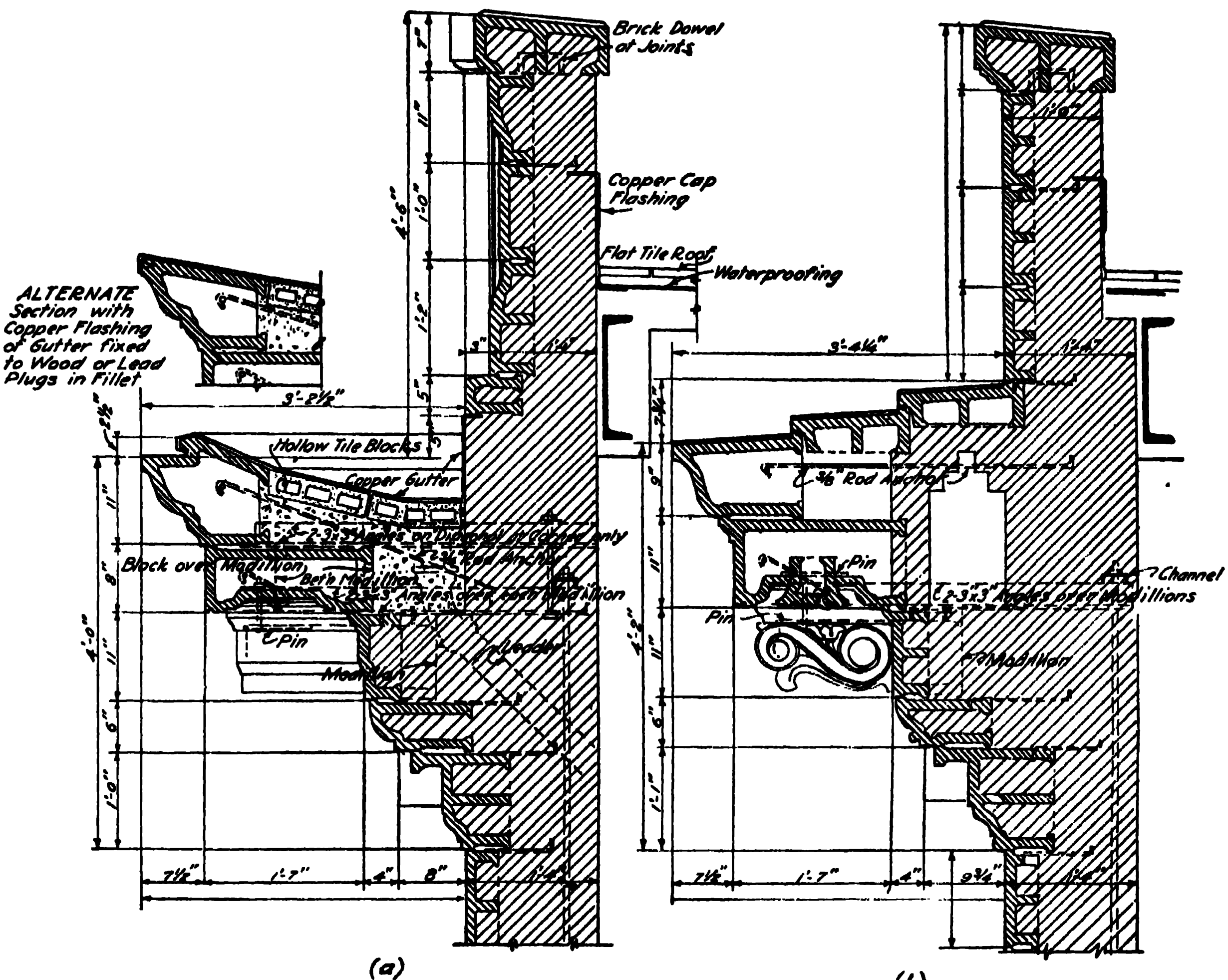


FIG. 475. TERRA COTTA CORNICES

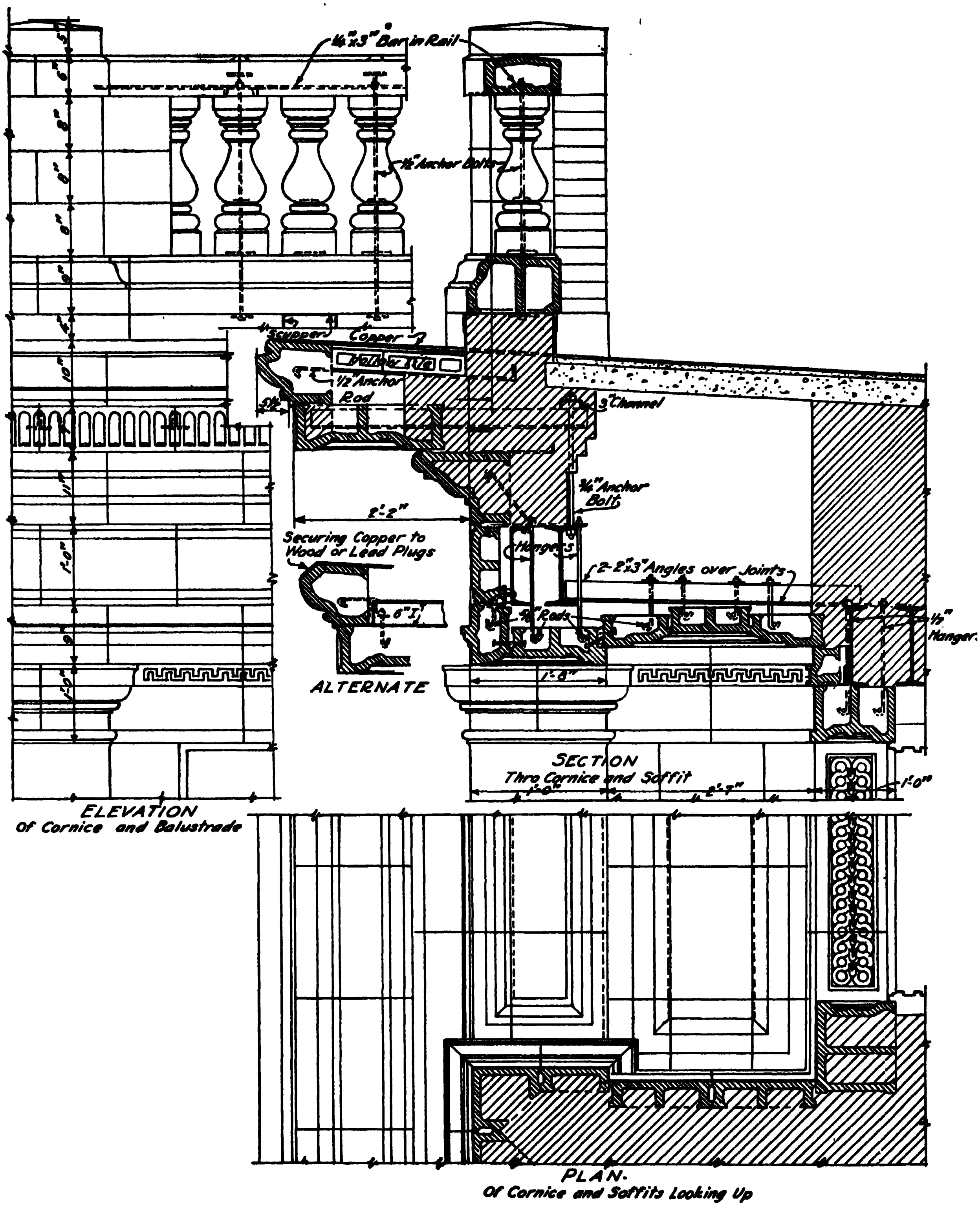


FIG. 476. TERRA COTTA PILASTERS

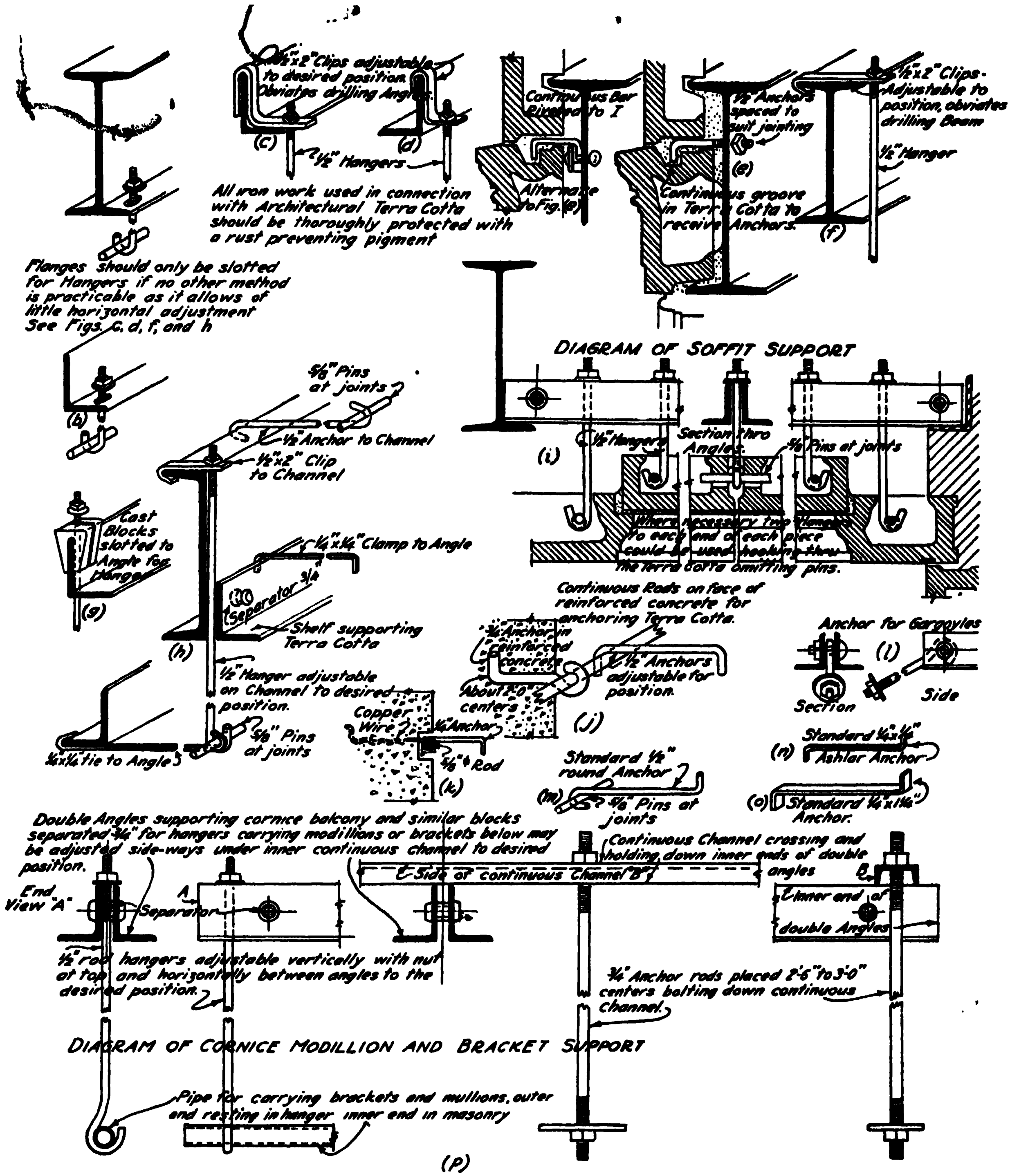


FIG. 477. TYPES OF ANCHORS

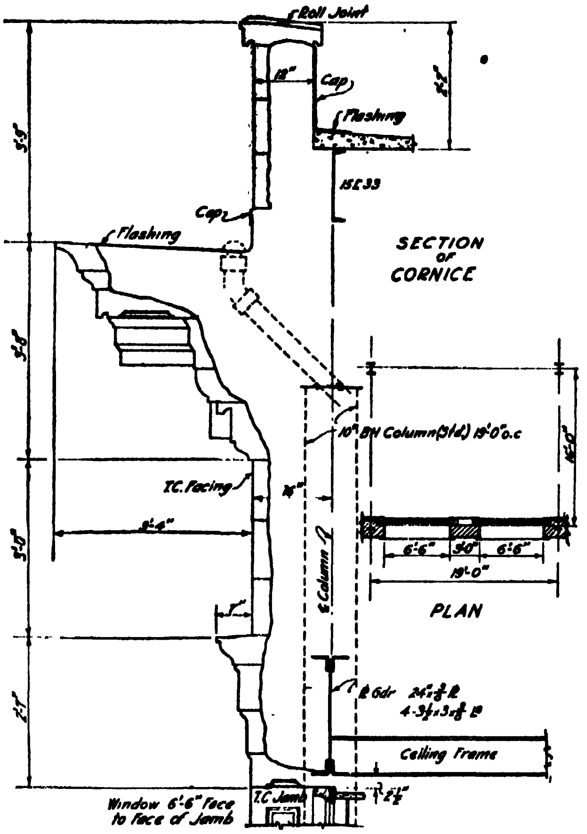


FIG. 478

Concentration at second pair of angles

Block D = 0.5 × 0.7 = 0.35 c.f. @ 100 =	35#/ft.
Brick work ¼ c.f. @ 120 =	30
Top of cornice 1.0 × 68 =	68
T.L. =	163
163 × 6.33 =	1030#

Concentration at angle next to parapet

0.5 × 68 × 6.33 = 215#

Figure 479 shows a loading sketch for the typical outlooker.

Moment in cantilever KL

1630 × 18 + 950 × 12 = 40,800″#

$$\frac{I}{c} = \frac{40{,}800}{16{,}000} = 2.55''^3 \qquad \text{2-4 ⊐ ⊏ } 5\tfrac{1}{4} \text{ O.K.}$$

Reaction at L = 1630 + 950 + 1030 = 3610#

Stress in LN = 3610 ÷ 0.707 = 5100#

$$\text{Min. } r = \frac{2.83 \times 12}{120} = 0.28'' \qquad 2 \text{ ∟ } 2\tfrac{1}{2} \times 2 \times \tfrac{1}{4} \text{ O.K. for knee braces.}$$

Uplift at M. Taking moments about L,

(1630 × 1.5 + 950 × 1.0 − 215 × 1.0) ÷ 2 = 1590#

Horizontal stress in LM = horiz. comp. of LN = 3610#

Resultant stress at $M = \sqrt{(1590)^2 + (3610)^2} = 3940\#$

The required number of rivets at points L, M, and N may now be calculated. Not less than three should preferably be used at joints of this kind.

Angles supporting book tile. Span = 6′-4″, spacing = 1′-0″.

Ld. per □′ from top of cornice	=	68
Block D (see above calc.)	=	42
Brick work (see above calc.)	=	40
Total	=	150# × 1.0 = 150#/ft.

$$\frac{I}{c} = \frac{1.2 \times 150 \times (6.33)^2}{16{,}000} = 0.45''^3 \qquad 2 \text{ [illegible] } 2 \times \text{[illegible]} \times \tfrac{1}{4} \text{ O.K.}$$

$$\left(\frac{I}{c} = 0.50''^3\right)$$

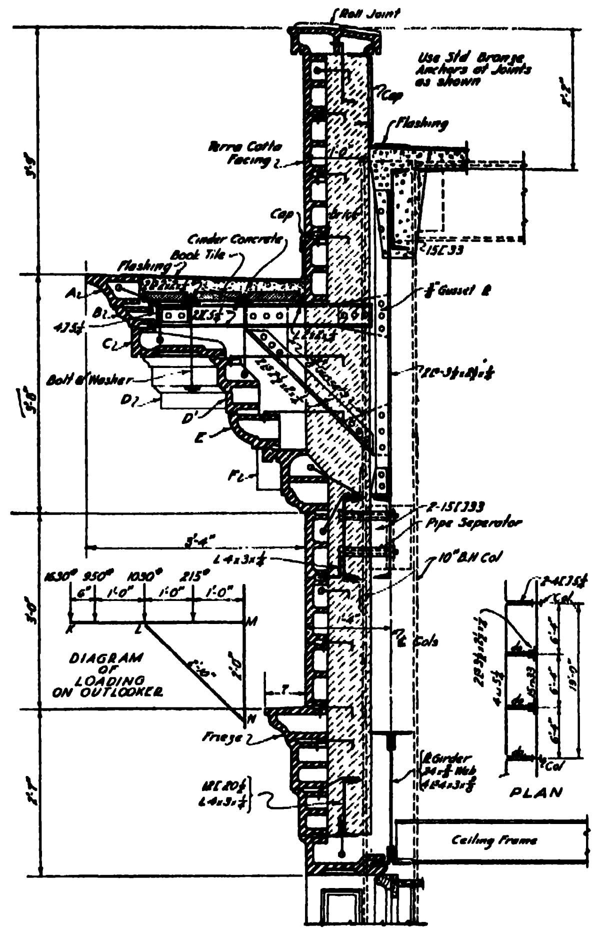


FIG. 479

Box girder supporting struts.

This also carries the wall and cornice not supported by the outlookers.

Parapet = 2.17 (¾ × 120 + ¼ × 100)	= 246
Wall from roof line down to outlookers	
(3′-9″ + 4″) − 2′-2″ = 1′-11″ high	
1.92 (¾ × 120 + ¼ × 100)	= 218
Brick work from outlookers down to girder = (3′-8″) − 4″ = 3′-4″ high average thickness = 16″	
3.33 × 1.33 × 120 =	533
Terra cotta (blocks E and F)	
1.5 × 0.66 × 100 =	100

Forward	=	1097
Load by shelf angle		
$1.33 \times 0.33 \times 100$ terra cotta	=	44
$1.33 \times 0.17 \times 120$ brick	=	27
Girder (assumed)	=	70
Total	=	1238#/ft.

Concentration from strut = 3825#

Girder loaded with two concentrated loads at third-points and the uniform load.

$$M = \frac{P \cdot L}{3} = \frac{3825 \times 19}{3} = 24{,}200$$

$$M = \frac{w \cdot L^2}{8} = \frac{1238 \times (19)^2}{8} = 55{,}900'\#$$

$$70{,}100'\#$$

$$\frac{I}{c} = \frac{70{,}100 \times 12}{16{,}000} = 52.6''^3$$

Use 2-15 [ ] 33

Spandrel at frieze.

Terra cotta (ave. thickness = 6″, 4′-6″ high)

$0.5 \times 4.5 \times 100$	=	225
Brick work $0.67 \times 4.5 \times 120$	=	360
Spandrel	=	15
Total	=	600#/ft.

$$\frac{I}{c} = \frac{1.5 \times 600 \times (19)^2}{16{,}000} = 20.2''^3 \quad \text{Use } 12 \text{ [} 20\tfrac{1}{2}.$$

302. Parapet Walls.

SPECIFICATION CLAUSE

All exterior walls greater than 20′-0″ high, except when finished as cornices, gutters, or crown mouldings, shall have parapet walls the full thickness of the wall proper, extending at least 3′-0″ above the roof and shall be coped.

Parapet walls extend above the roof line as specified above, but generally they carry no load except that they may support a tank or some other special equipment. It is of course necessary to allow for their weight, which is usually carried by beams at the roof level. Party walls are also generally specified as above. Provisions must also be made for curb walls around skylights. Figure 480 illustrates typical details.

by their own foundation walls. In other instances, the bay window treatment may begin at some floor above the ground (usually the second floor) on account of storefronts in the first story or the proximity to the sidewalk. The latter cases require special framing to support the walls of the bay and the floors at such points. Some form of canti-

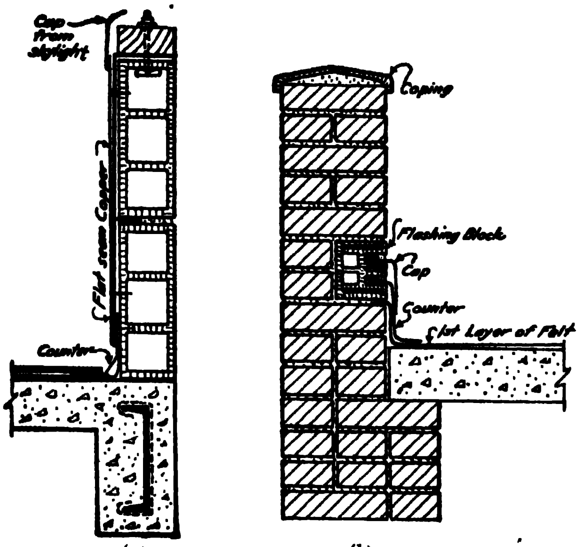


FIG. 480. FIRE WALL COPING
(*a*) copper on tile skylight curb wall
(*b*) brick parapet with flashing blocks

lever is usually involved. The bays may be carried at each floor, or the whole tier may be supported at the bottom. The choice of the method will depend upon the nature of the framing at each floor, available headroom, and so on.

The support may be provided by either cantilevering some of the interior floor beams out to receive the bay window framing, or by running a header beam across the bay, from which a bracketed frame

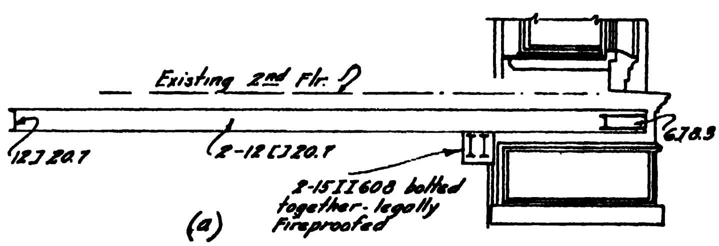


FIG. 481 (*a*)

303. Cantilevered Bay Windows.

In certain types of office or apartment buildings, bay windows are introduced in the front elevations. In some cases, these may extend from the roof down to the ground, where the walls may be carried

may be built. The first method requires the beams to be framed into a girder which can absorb any uplift produced, while the second method requires the header beam to be anchored down against uplift.

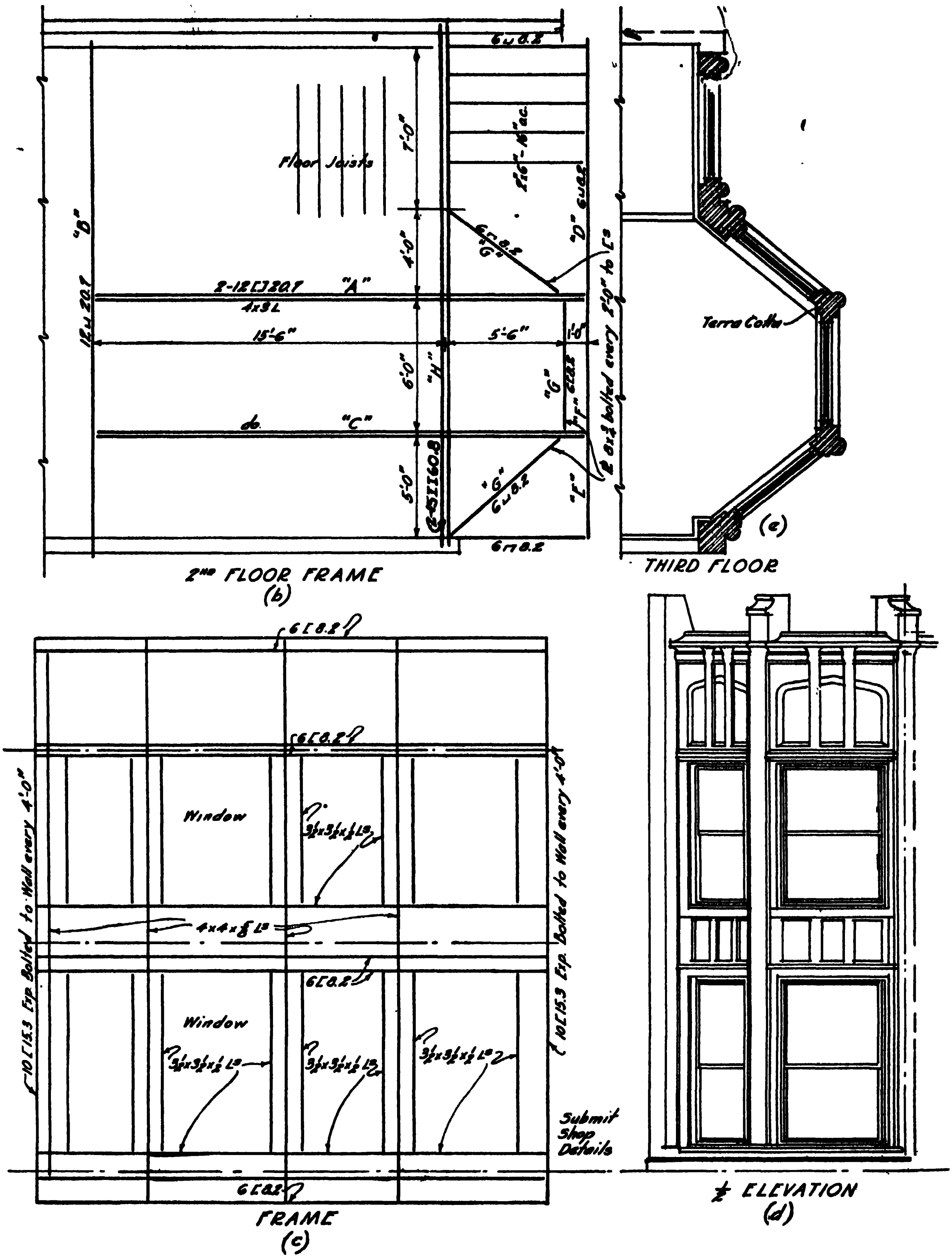

Floor Joists
2-12[]20.7 "A"
4x3L
15'-6"
5'-6"
1'-0"
do. "C"
7'-0"
4'-0"
6'-0"
5'-0"
"H"
"B"
"D" 6∟8.2
"G"
"E"
"F"
6∟8.2
6[8.2
Terra Cotta
(e)
THIRD FLOOR
2ND FLOOR FRAME
(b)
6[8.2
Window
4x4x5/8 Ls
Window
3½x3½x½ Ls
10[15.3 Exp. Bolted to Wall every 4'-0"
Submit Shop Details
FRAME
(c)
½ ELEVATION
(d)

Fig. 481 (b), (c), (d), (e).

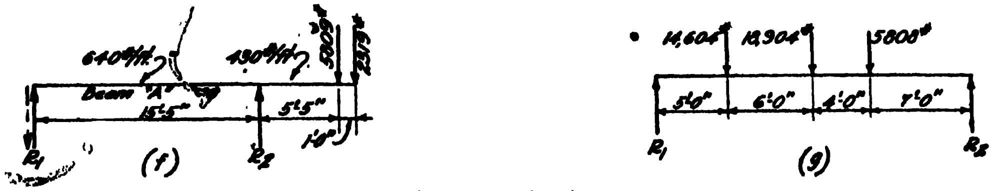
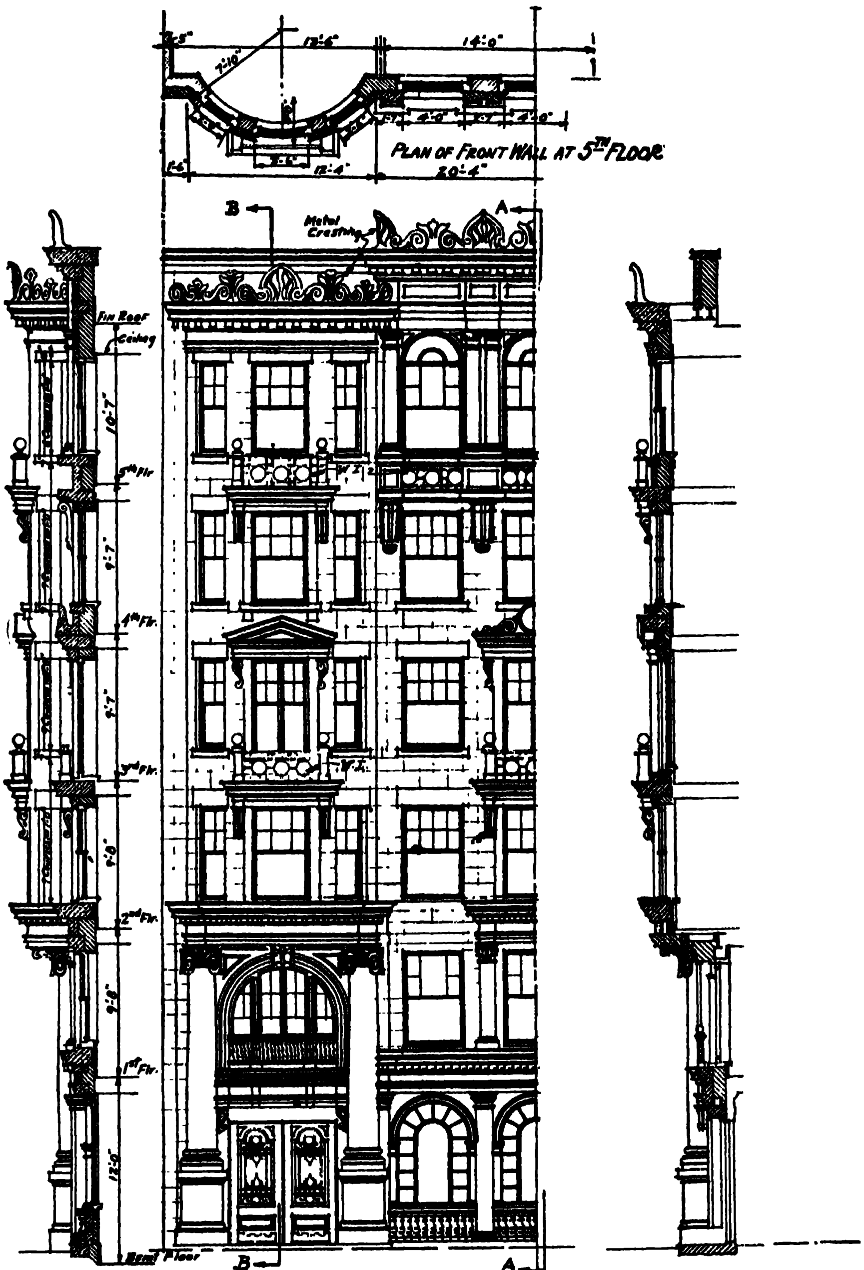


FIG. 481 (f), (g).



SECTION A-A FRONT ELEVATION SECTION B-B

FIG. 482

Figure 481 shows one method of supporting a bay window. In this particular case, the beams "*A*" and "*C*" in (*b*) were extended far enough back so that only downward reactions occurred on the girder "*B*." The terra cotta walls and the window frames are supported by angle framing as shown, and the local floor loads by the channels framing into the struts.

Illustrative Prob. 303a. Check the typical sizes shown in Fig. 481.

Roof over store (below second floor)
T.L. = 60#/□' Span = 6'-0"
Use 2" × 6" spruce — 16"o.c.

Beam "*A*" T.L. floor = 70#/□'.
On anchor span 8.5 × 70 = 595
Beam = 45
640#/ft.

On cantilever 5.5 × 70 = 385
Beam = 45
430#/ft.

Concentration from channels "*G*" $= \frac{5809}{2} \times 2 = 5809\#$

" " " "*D*" and "*F*"

$$= \frac{4229}{2} + \frac{409}{2} = 2319\#$$

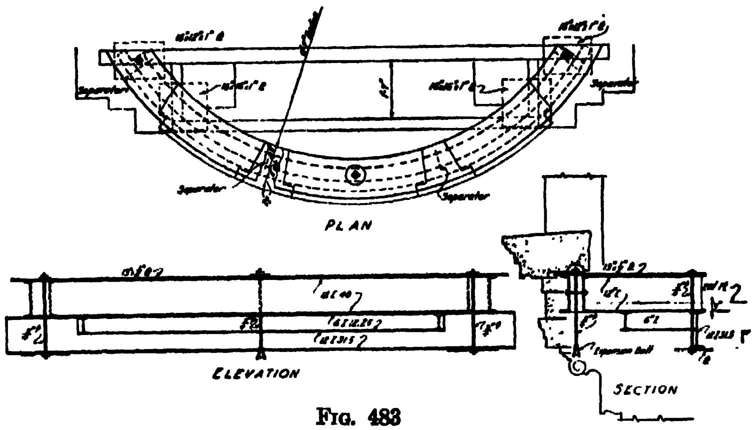


FIG. 483

Channel "*D*" span = 11'-0"
Load per foot = 3 × 60 = 180#
180 × 11 = 4180#
6 × 8.2 = 49
4229# total — Use 6 [8.2

Channel "*E*" 5 × 3 × 60 = 900 — L = 5'-0"
5 × 8.2 = 41
941# total load — Use 6 [8.2

Channel "*F*" 6 × 60 = 360 — L = 6'-0"
6 × 8.2 = 49
409# total load — Use 6 [8.2

Height of bay = 24'-0" Average wt. = 40#/superficial foot
Channels "*G*" 24 × 40 × 6 = 5760# — L = 6'-0" ±
6 × 8.2 = 49
5809# total load
Use 6 [8.2

Figure 481 (*f*) shows a loading diagram.
R_1 = 1507# and R_2 = 18,904#
M_{max} occurs at R_2 = 53,514'#

$$\frac{I}{c} = \frac{53{,}514 \times 12}{16{,}000} = 41.1''^3$$ Use 12] [20.7

Figure 481 (*g*) shows a loading diagram for girder "*H*."
R_1 = 24,830# and R_2 = 16,745#
M_{max} = 173,060'#

$$\frac{I}{c} = \frac{173{,}060 \times 12}{16{,}000} = 129.75''^3$$ Use 2-15 I I 60.8

Figure 483 shows another method of supporting a bay window. In this, the uplift is taken care of by anchoring the header beam into the wall. This is an inadvisable scheme if other means may be used. Figure 482 shows a partial elevation and sections of supported bay windows.

Prob. 303b. Check the typical sizes shown in Fig. 483.

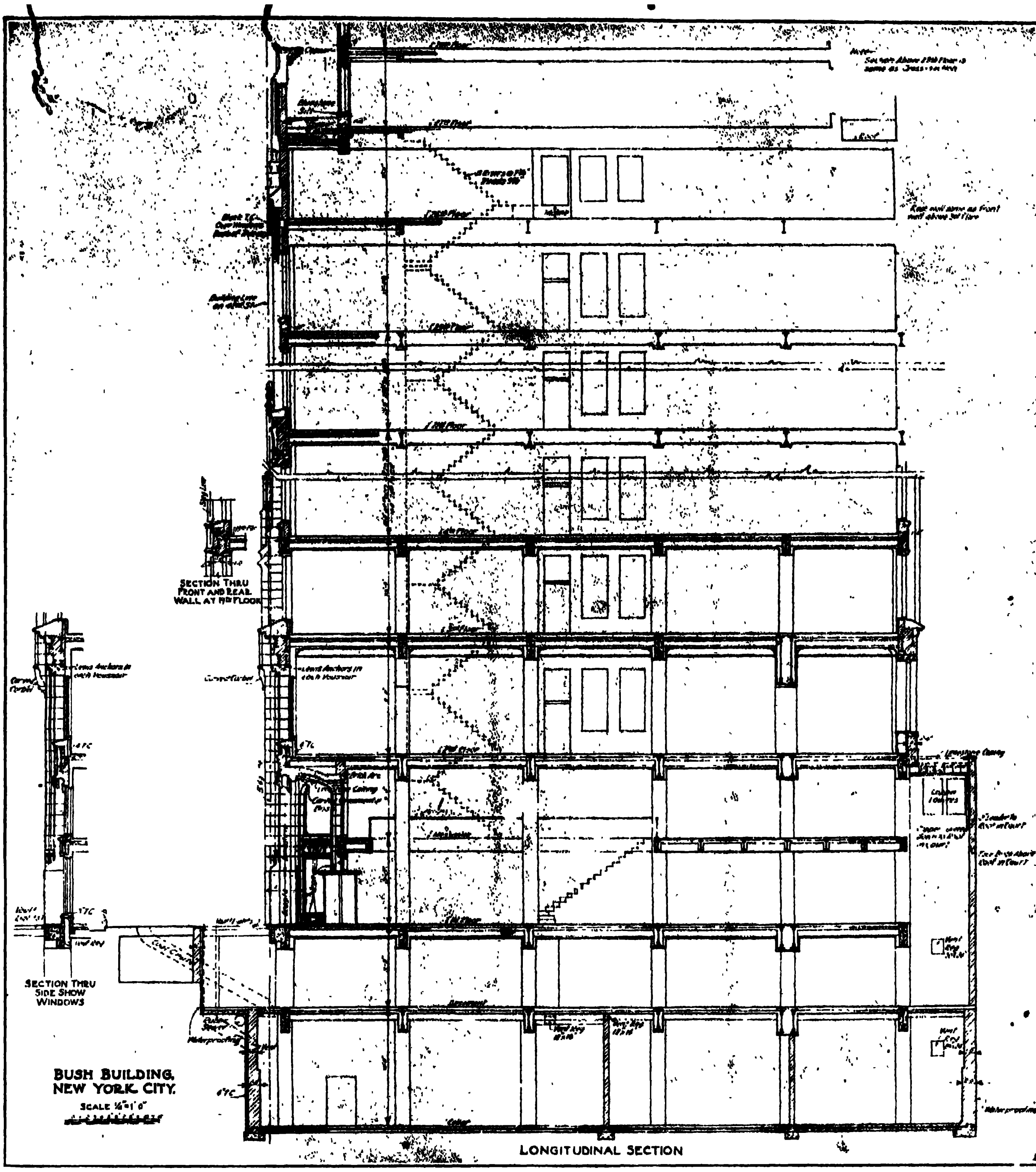


PLATE 34 THE OFFICE BUILDING
LONGITUDINAL SECTION
HELMLE & CORBETT, ARCHITECTS

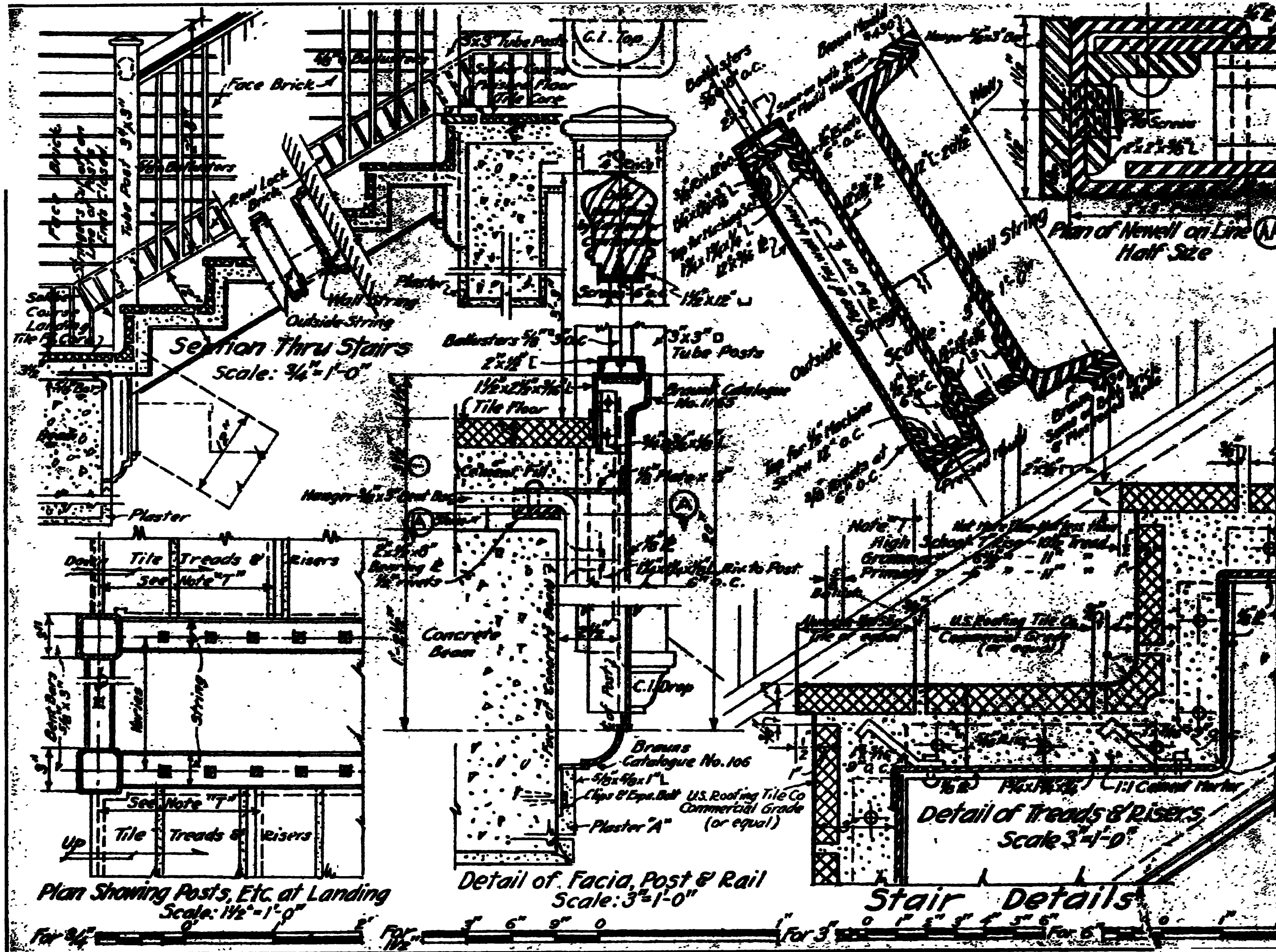


PLATE 35 THE SCHOOL
TYPICAL STAIR DETAILS
WALTER R. McCORNACK, ARCHITECT

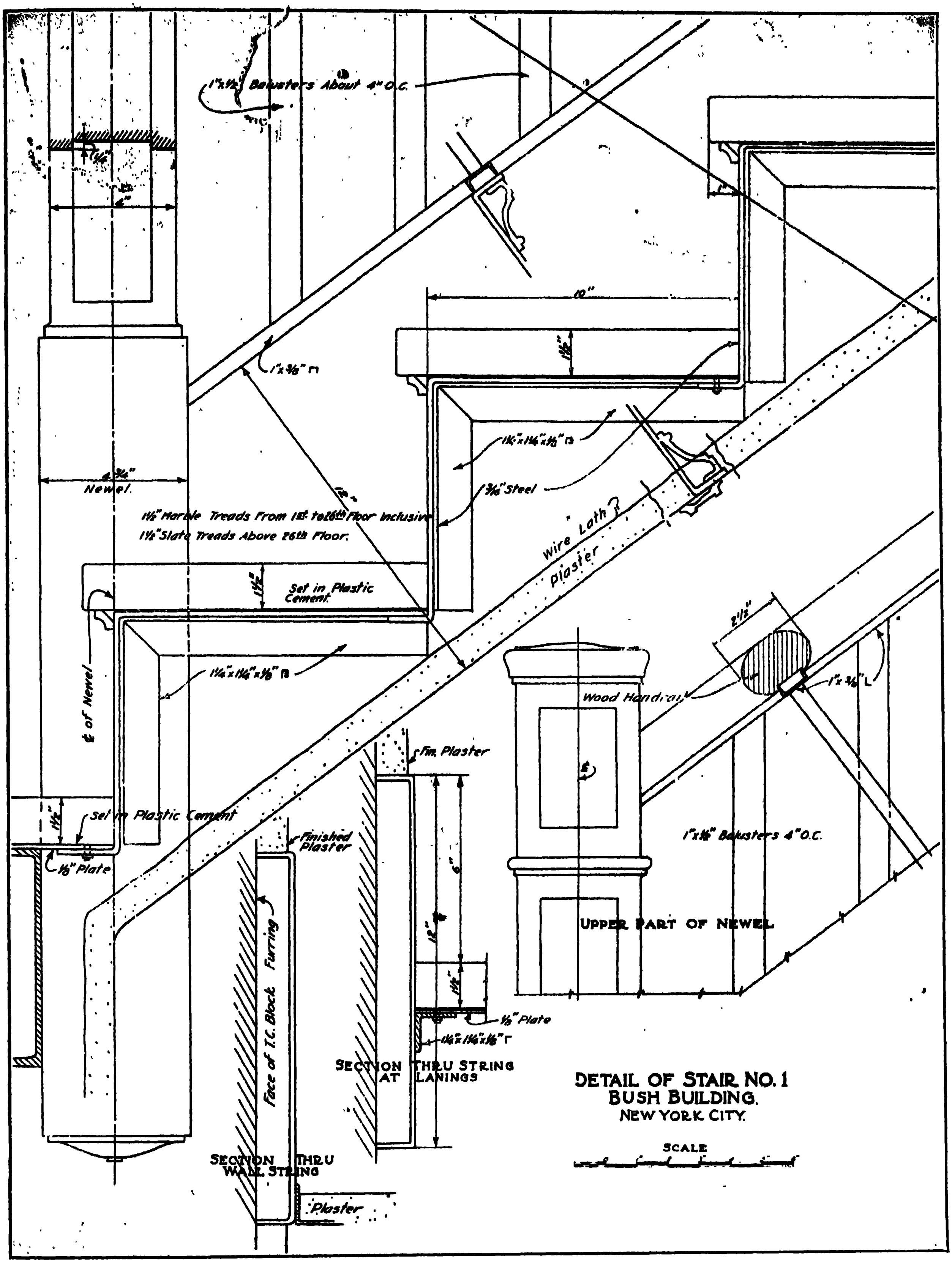


PLATE 36 THE OFFICE BUILDING
STAIR DETAIL
HELMLE & CORBETT, ARCHITECTS

CHAPTER 26

STAIR CONSTRUCTION

304. General.

Laying out and arranging stairs is usually the province of the architect, but the structural engineer is sometimes called upon to proportion stairs, particularly for mill buildings and the like, which will be economical of space and materials.

Figure 484 gives diagrams of the various types of stairs, as well as the definitions of rise and run and the discrimination between tread and run. Ordinary stairs have an inclination of from 35° to 40° with the horizontal. Public buildings often have much easier stairs, while unimportant stairs present a slightly steeper inclination. For 10° or less, ramps are preferable, with 15° to 18° as the limit of slope. For inclinations exceeding 50°, right and left stairways* are sometimes used. For angles of 70° or more, ladders are required.

There are many considerations in a carefully planned stairway, such as an **easy rise** and **ample head room.** The latter should never be less than 6′-6″ and 7′-0″ is a preferable minimum. A section, as well as a plan, should always be drawn to check up the clearances. Figure 485 shows such a layout for a simple stairs in a residence

Winders should be avoided, especially in public

* In these, each step is only half the width of the stairway, and alternately for the right and left foot will usually be preferable to a stairway in which each step overhangs the one below.

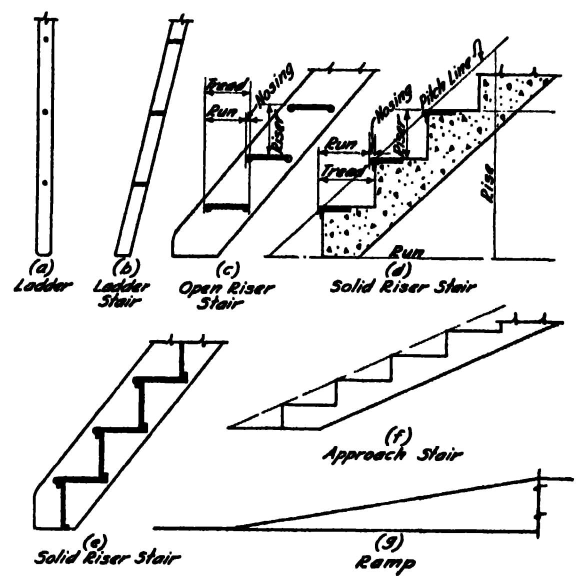


FIG. 484

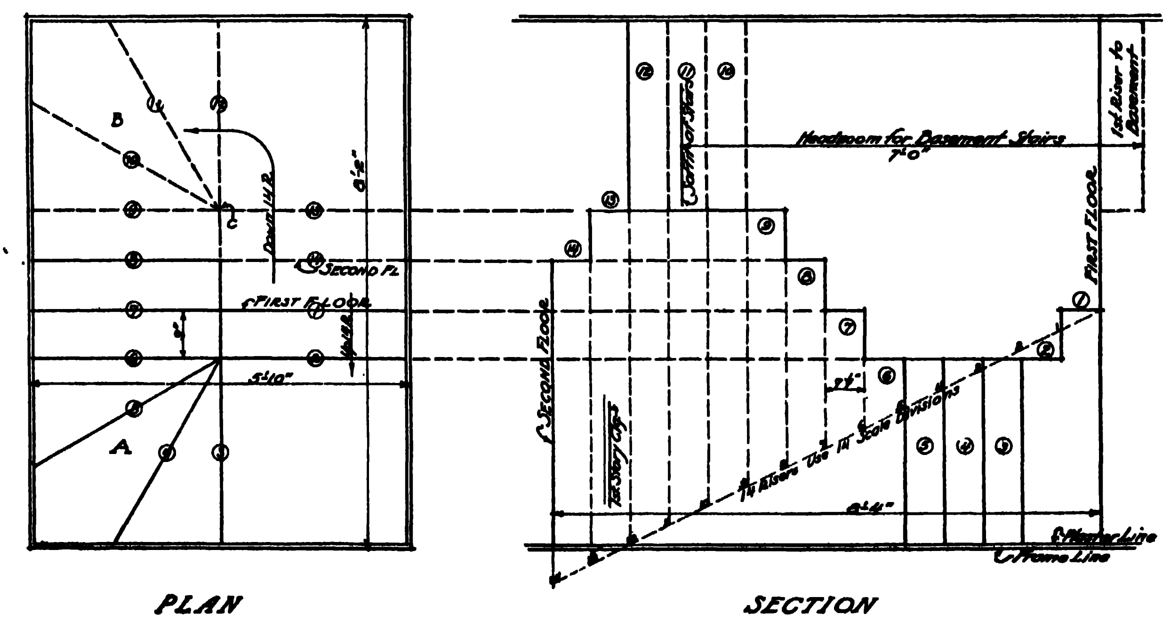


FIG. 485. LAYING OUT A STAIR

buildings, and many building ordinances prohibit their use. If employed, wider treads should be used, so that the tread at a distance of about 16″ inside the inner rail is as wide as the ordinary tread. The number of risers in one run should be limited, and some codes restrict this to 15. The minimum number of risers between landings should be 3. All the risers and treads in the same flight should be the same height and width respectively, even if risers in fractional inches result. The riser heights should not change appreciably from story to story.

The risers usually vary from 6″ to $7\frac{1}{2}$″, 7″ to $7\frac{1}{2}$″ being common. They should never exceed 8″ and even $7\frac{3}{4}$″ makes an uncomfortable rise for continual use. The treads vary from 9″ to 12″ and are proportioned according to the rise, — the less the rise the greater the tread. Nosings are usually from $1\frac{1}{4}$″ to $1\frac{3}{4}$″. Where none are used, the treads should be made slightly wider than otherwise.

The width of stairs varies with the use and the type of building. Ordinances often specify minimum requirements of this kind. Common widths of stairs range from 2′-6″ to 4′-0″, 3′-0″ being a value often used. This feature should be carefully studied in the code controlling the design in question. The Chicago code gives a very typical example of stair width regulations in mill or slow-burning construction and is as follows:

SPECIFICATION CLAUSES

With floor area of 6000 square feet or less, two stairways.

With floor area of 6000 to 12,000 square feet, three stairways.

The width of stairs required in buildings of mill or slow-burning construction shall be computed as follows:

The width of stairs in inches shall be equal to the result obtained by deducting 3000 from the floor area of the building in square feet and multiplying the remainder times eight and dividing the product by 1000 and adding 72 inches to the quotient, expressed in the formula as follows:

$$72 \text{ inches plus } \frac{(\text{area} - 3000) \text{ times } 8}{1000}.$$

For other types of construction, the following* is a typical example:

SPECIFICATION CLAUSES

When exit doorways have a clear width of at least 40″ each, the aggregate widths of such doorways shall be equal to the required width of corridor or stairway served by same. When individual doors are less than 40″ wide, there shall be one doorway for each 22″ of required width of corridor or stairway leading to same. Every doorway shall be at least 28″ wide in the clear. All passageway exit doors shall swing in the direction of exit travel, except in case of horizontal exits where direction of travel may be indeterminate.

All exit doors leading from rooms having an occupancy of 15 or over, shall open in the direction of exit travel, except in schools where fire drills are organized under control of the teachers.

Note. — In schools where pupils are trained in fire drill, it is considered essential that classroom doors should open inwards, as they are better adapted to positive control by the teachers in time of panic. If necessary, a teacher can back against a door and hold it until the pupils are in marching order, and the proper time arrives for them to file out.

The opening of one door shall not be permitted to obstruct another, and the arc of opening of doors which open upon stairway landings or platforms shall not reduce the width of the passageway to less than the required width of the stairs.

Every room having an occupancy of more than 75 persons shall have at least two doorways remote from each other leading to exits.

Hallways or corridors at the street or court level furnishing exit from stairways, shall be not less in width than the aggregate width of the required stairways which they serve. Every hallway or corridor which may serve as an exit for 50 or more persons, shall have at least 44″ of width for the first 50 persons and 6″ additional for each additional 50 persons to be accommodated thereby. This computation shall be based on the number of persons in the story having the largest occupancy served by said corridor.

Stairways used as required means of exit shall be at least 44″ wide between faces of walls, or 40″ wide between face of wall and an open balustrade, or between two open balustrades. All such widths shall be clear of all obstructions except that hand rails attached to walls may project not more than $3\frac{1}{2}$″ within them, or stringpieces more than $2\frac{1}{2}$″. If newels project above tops of rails, a clear width of at least 44″ shall be provided between the faces of the newel and the face of the wall or newel opposite. All stairs shall have walls or well secured balustrades or guards on both sides, and except in dwellings, shall have hand rails on both sides. A stairway of 7′-0″ or more in width shall be provided with a continuous intermediate hand rail substantially supported. All stairs shall have treads and risers of uniform width and height throughout each flight; the rise shall be not more than $7\frac{3}{4}$″, and the tread exclusive of the nosing not less than $9\frac{1}{2}$″. Stairways exceeding 12′-0″ in height shall have an intermediate landing.

Note. — For stairways in primary schools it may be advisable to reduce the height of risers here given.

In determining the occupancy of any building, the width of stairways required for any floor area above the first floor shall be determined by the number of persons occupying such floor area, computed on the basis of fourteen persons

* Excerpted from "Building Code" of The National Board of Fire Underwriters, New York.

> for each 22" width of stairway, plus one person for every 3 square feet of hallway floor and stairway landings in the story height of such floor, excepting that in any building where a system of automatic sprinklers is installed throughout the entire building, the number and width of stairways may be computed on the basis of twenty-one persons for each 22" width of stairway; and excepting that when horizontal exits are provided, the number and widths of required stairways for floor areas above the first floor may be diminished to a basis of fifty persons for each 22" width of horizontal exit.

305. Rules for Proportioning Risers and Treads.

There have been a number of ordinary rules for proportioning risers and treads in stairs, and good results are obtained where the risers vary from 7" to 7½" with such rules. The first used was that the riser plus the tread = 18", based upon an 8" riser and a 10" tread. This gave too hard a stairway and it was modified to 17" or 17½". By changing the riser or tread 1" either way, a hard stairway resulted, so that a rule of twice the rise plus the tread = 23" to 25" was used. This gives a more balanced stair than the previous rules. Another rule sometimes given is that the product of the rise and run should be from 66" to 75", generally from 72" to 75". This allows greater variations. Summarized, these are often given as:

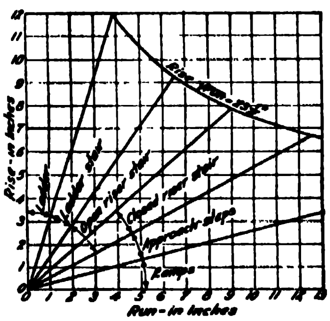


FIG. 486

$$\text{Rise} + \text{run} = 17'' \text{ to } 17\tfrac{1}{2}'',$$
$$2\,(\text{rise}) + \text{run} = \text{not} < 24'' \text{ and not} > 25'',$$
$$\text{rise} \times \text{run} = \text{not} < 70 \text{ nor} > 75.$$

A rule* which is the result of further study is rise $\times \sqrt{\text{run}} = 23\frac{1}{2}$. Figure 486 shows these relationships and suggested limitations for various types.

"The ordinary rules for proportioning risers and treads are satisfactory for the common cases, but steep stairs result for the higher values of the rise, and shallow stairs for the lower values of the rise. The rules are not particularly well adapted to special cases, and in many cases designers have had to resort to judgment in such instances, such as for public buildings and outside approaches. Recent rules have been evolved which cover all cases, based upon the study of the lengths of steps." † These are based upon a ratio of horizontal to vertical step of 2.4 to 1.0, and are as follows:

$$t = s - 2.4\,r, \text{ and}$$
$$r = \frac{s - t}{2.4}, \text{ in which}$$

s = the horizontal step,
t = the tread, and
r = the riser.

The values of the basic horizontal step, s, are selected according to the type of building. The following are representative:

Power houses, factories (men employees), office building emergency exits	28.5" to 30.5"
Factories (men and women employees), public buildings, railway stations, office building regular stairs, average hotels and ordinary residences	26.5" to 28.5"
Theatres, department stores, high-class hotels and residences, outside steps for public buildings	24.5" to 26.5"
Primary and grammar schools	23.5" to 24.5"

Occasionally for outside steps, the treads are made wide enough to take two steps on each tread. The rise and tread in such cases should be so proportioned that the additional width is a mean between the single tread and the basic horizontal step. The following formula applies:

$$d = 25 - 3.6\,r, \text{ in which}$$

d = the width of the double tread.

Figure 487 shows several types of stair layouts. The selection of course depends upon the surrounding conditions. The plan and section should always be worked out together, so that headroom and the relations of the landings to windows and floors may be studied.

306. Structural Steel Framing for Stairs.

The relation of the structural steel contract to the miscellaneous iron contract is often confusing in connection with stairs. It is a customary procedure for the structural steel contractor to furnish only the main frame for the wells, including the header beams at the floor grades. When "straight-run" stairs occur, the header beam at the landing is sometimes made a part of the structural steel contract also.

The structural engineer is therefore primarily interested in defining and locating only the frame which forms the well. Figure 488 gives a typical illustration of this kind (see also floor openings on Pl. 22). The header beams (and landing beams if provided) are usually made 10 [15.3. Such a size ordinarily is sufficient for the average loads and span. In some cases, a smaller size could be used, but the 10" size is employed as a minimum in order to provide sufficient depth for the connection of the stair stringers. The size should of course be checked for the actual loading conditions. When channels are used, they are generally faced so that the backs

* Given by Mr. Raymond C. Reese, Pittsfield, Mass., and developed from the Cleveland Building Code, in Engineering News Record, Vol. 86, p. 805.

† By Mr. Charles H. Nichols, M.E., Vice-President of the Goff Engineering Corporation, Consulting Engineers, New York City.

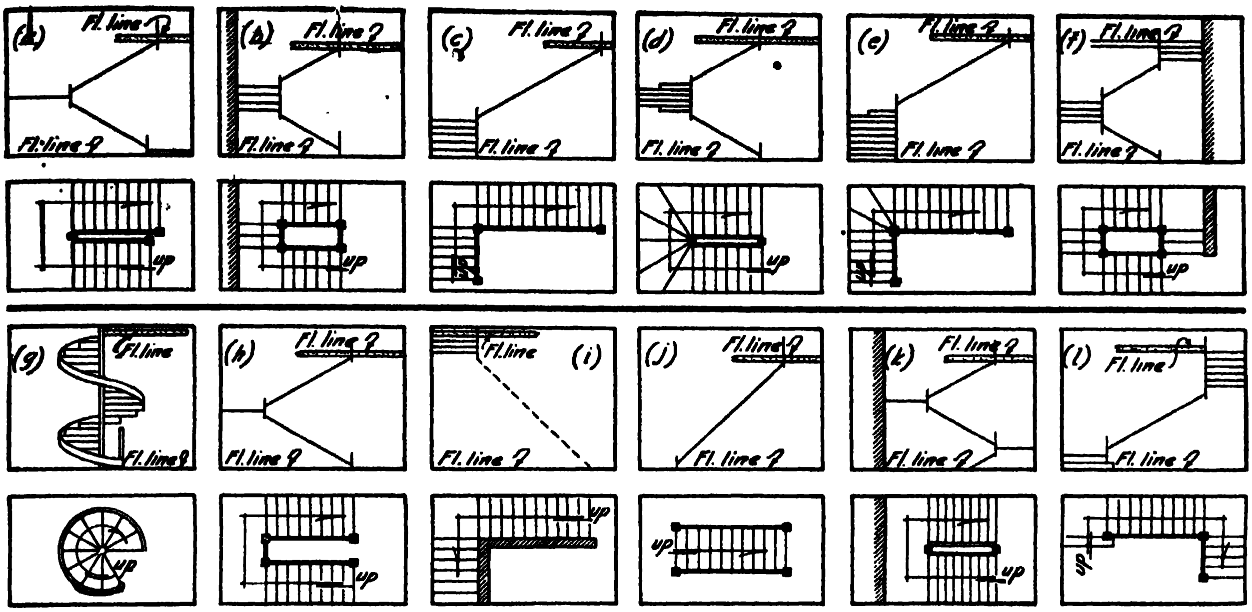


FIG. 487

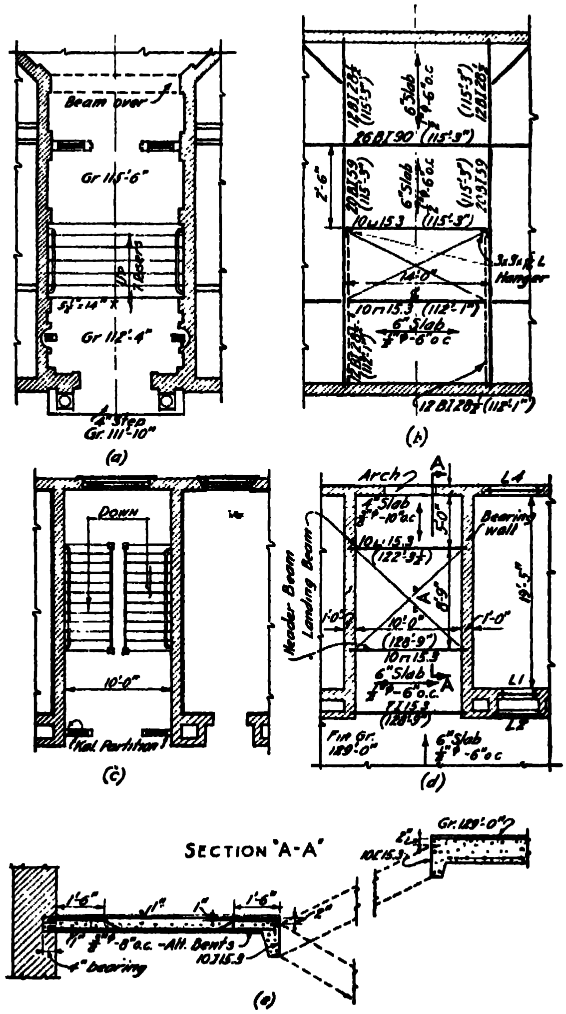


FIG. 488

are toward the stair run, as shown in Fig. 488 (*e*). This allows simpler stringer connections.* Some engineers prefer to use I beams for such headers, as it is difficult to obtain good details for fireproofing the backs of channels. However, I beams require coping and blocking of the stair stringers in many cases, in order to make the connections. As a 10″ minimum depth should be used, the use of I beams means an excess of steel above that in reality required.

The dimensions locating the framing beams should be given on the structural plans. The dimension "*A*" in Fig. 488 (*d*) is determined by the number of treads in the run and their width, and the relation of the header beams to the faces of the risers. This requires reference to the architect's plan and section of the stairs, and to the architectural stair details. Plate 35 gives a typical illustration of the latter. Further details are illustrated in Pl. 36. The width of the landing is fixed by the architect, so that the landing beam may be located also, as shown in Fig. 488. The wall opening is indicated by light diagonal lines, as shown. This not only shows an opening at the plane of the frame, but conveys to the structural steel contractor that he has no material inside of it to furnish. In special cases, it may be desirable from a construction point to have the landings a part of the general contract in order to expedite other work. For a flight of steps such as shown in Fig. 488 (*a*) it is necessary to provide a hanger to support one of the beams as shown in (*b*). This is made a part of the structural steel contract. Except in special cases like this, the light iron contractor supplies any required hangers and fittings.

The design of stair-supporting, structural beams involves no new methods. With the unit load per sq. ft. established, the load per running foot on a beam may be determined. This is usually based upon an average dead and live load per sq. ft., and the **tributary area in horizontal projection.** The live load for stairs from large gathering places, such as armories, assembly halls and so on, is usually specified as 100#/□′. For all other stairways, 75#/□′ is common† (except in small residence work). The dead load will depend upon the type of treads and risers used. In general, **a total dead load of 50#/□′** is a conservative allowance. The average weight of the steel in the sub-tread construction itself is about 6#/□′. If the stair soffit is plastered (as shown on Pl. 36), an allowance of 10#/□′ should be made in addition.

* This also allows the flange to give its support to the floor or landing tributary to the channel and a clear vertical surface to attach the stringer connection angles to at any reasonable elevation.

† This value is based upon the reasoning that a person occupies about 2′-0″ of tread, and with a 150# average weight, this is equivalent to 75# per linear foot of step.

Illustrative Prob. 306a. Design the stair header beam at the landing in Fig. 488 (*c*) and (*d*).

Landing		*Stairs*	
L. L. =	75	L.L. =	75
1″ mastic =	12	Composition treads =	12
4″ slab =	50	Pressed steel sub-treads and risers =	6
Susp. ceil. =	15	Stringers, railing, etc. =	2
T.L. =	152#/□′	Plastered soffit =	10
		T.L. =	105#/□′

From landing 152 × 2.50 = 380
From stairs 105 × 4.38 = 460
Beam and fireproofing = 50

T.L. = 890#/ft.

L = 10′-6″ c.c. bearings

$M = 1.5 \times 890 \times 10.0 \times 10.5 = 140{,}000''\#$

$$\frac{I}{c} = \frac{140{,}000}{16{,}000} = 8.75''^3 \text{ required}$$

Use 10 ⊏ 15.3‡ on a/c of framing in stair stringers.

$$\frac{I}{c} = 13.4''^3 \text{ O.K.}$$

Prob. 306b. Check the size of stair header beam at the top of the stairs in Fig. 488 (*a*) and (*b*) if the L.L. = 100#/□′ (public entrance), 1″ granolithic finish in corridor, 1¼″ marble treads and 1″ marble risers. Stringers at third-points of opening which concentrate stair loads at third-points of header beams.

307. Steel Sub-tread Stairs.

While the structural engineer seldom has to design or detail any miscellaneous framing for stairs, he may be requested by the architect to approve details submitted, in a general way, or to check them for strength. Hence it is not unreasonable that he should understand the general features of such work.

In steel-framed buildings, the stairs are usually made of steel sub-tread construction, and occasionally, this is true for concrete-framed buildings. It is difficult to fireproof such a stairway completely, and still get an ornamental effect. However, the sub-tread construction offers partial protection, as the superimposed treads and risers may be made of fire-resistant material and thus protect the upper side of the steel, and the soffit may be plastered with cement plaster.§

With data such as is shown in Fig. 488 and on Pl. 35, the miscellaneous iron contractor takes over the furnishing and erection of stringers, intermediate landings, treads, risers and hangers. Plate 37 shows a typical shop drawing. The architect should be familiar with these types of drawings, as he must frequently check them over, at least to the extent of noting whether the details will carry out his plan. In drawings such as Pl. 35, any structural steel is often shown in dotted lines, while the stair frames

‡ Practical considerations very often control a choice in this work.

§ Some building codes require stair halls to be equipped with sprinklers in addition.

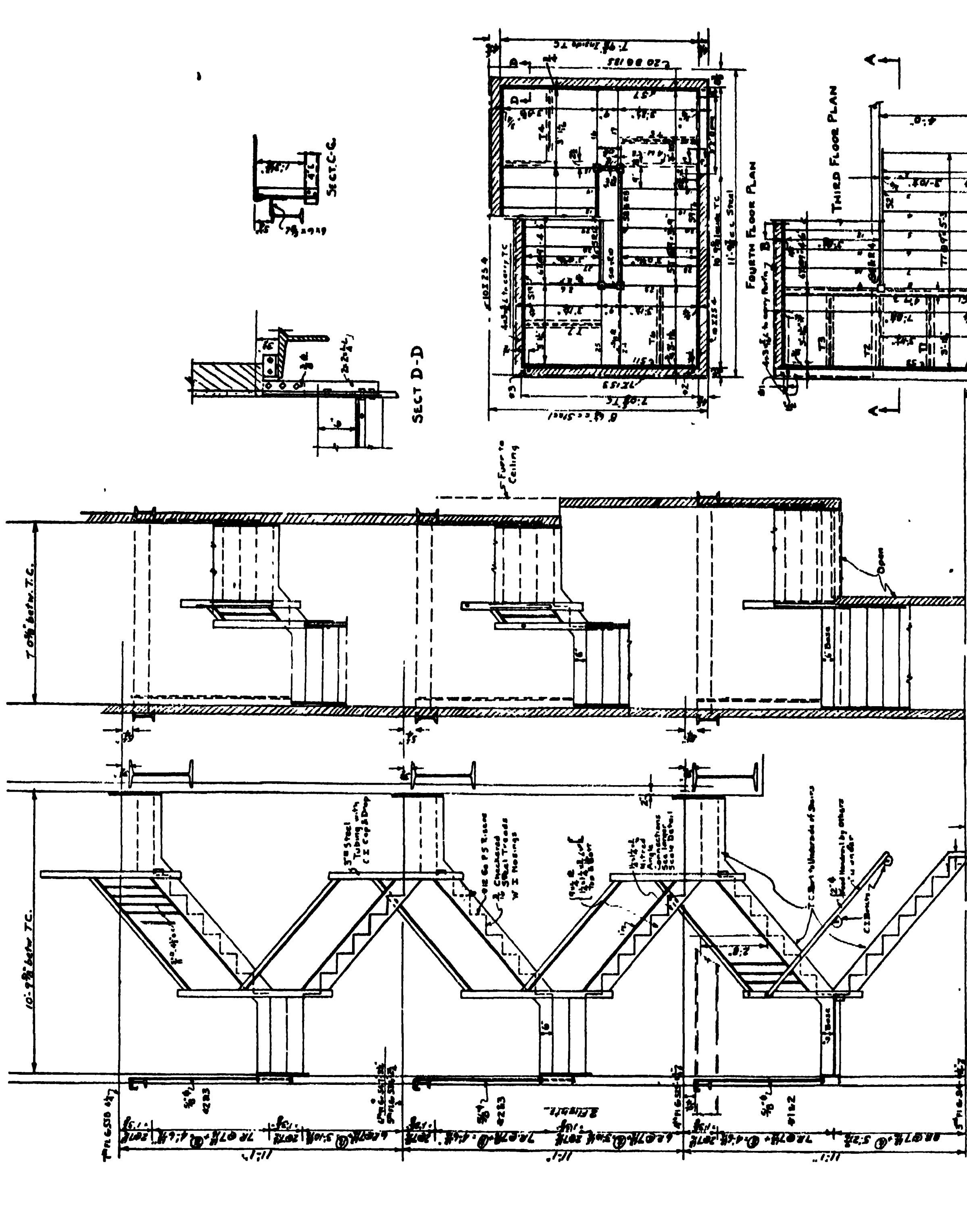


PLATE 37 TYPICAL SHOP ASSEMBLY DRAWING
STAIR FRAMING

and fittings are shown in full lines for clear differentiation. Figure 489 shows a typical stair installation. The members which carry the stair load to structural supports are called stringers. These may be made of pressed steel ($\frac{1}{8}''$ or greater in thickness), rolled channels, or built-up sections of thin plates and small angles. These are mentioned in order of their relative frequency of use. The pressed-steel stair is rapidly supplanting the old type of channel and plate sub-tread stair. The treads and risers are

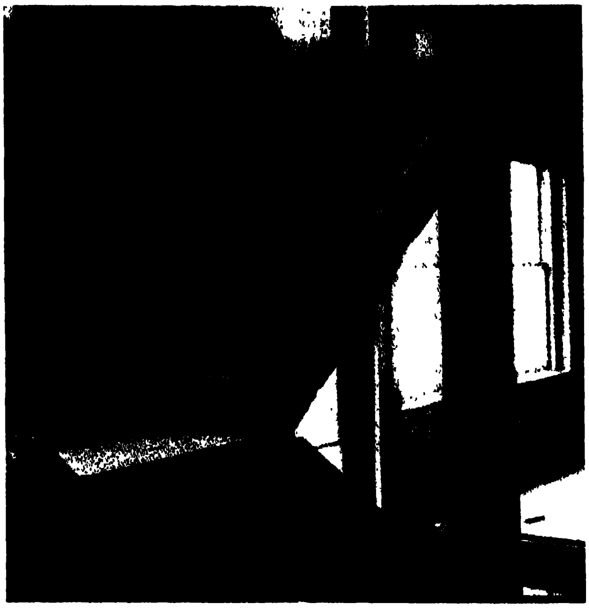


Fig. 489

commonly made of pressed steel parts, either in one piece or two, for the riser and the tread, and these are clipped to the stringers. The following is a typical specification:

SPECIFICATION CLAUSES

Stair No. 1 Stair No. 1 shall be built up of steel plate strings, 12″ × $\frac{1}{4}$″, facias of required depth, with ornamental steel mouldings attached for all face strings. The risers and treads shall be $\frac{3}{16}$″ steel, made up in a single unit, and bolted to steel angle carriers riveted to strings. The wall strings shall be plain with top and bottom flange, formed by turning back the edges of plate. Landings shall be formed by continuing wall and face string, finishing about 6″ above the level of floors and landings; landings and platforms shall be built up with proper members, to safely carry a live load of 100 pounds per square foot. Newels shall be cast 5″ × 5″ square, paneled, with moulded cap and drop. Rails shall be made up of $\frac{1}{2}$″ × 1″ balusters, three to a tread, top and bottom rails 1″ × $\frac{3}{8}$″ channel, prepared to receive moulded wood hand rail. All platforms and landings shall be provided with a steel plate, $\frac{1}{4}$″ thick, covering the entire area of said landings and platforms and bolted to the supporting ribs and strings.

Stair No. 2. Stair No. 2 shall be built up with plain 10″ × $\frac{3}{16}$″, with $1\frac{1}{4}$″ × $1\frac{1}{4}$″ angle, top and bottom, rails of $\frac{5}{8}$″ square, 5″ on centers, with top and bottom rail of 1″ × $\frac{5}{8}$″ channel. Finish the rail with a wrought iron pipe, $1\frac{1}{2}$″ internal diameter, with special globe fittings at each end, to return against cast iron newels. The newels shall have a base about 5″ square and the shafts shall be about $4\frac{1}{2}$″ × 2″, all as per details. The under treads and risers shall be No. 12 plate, arranged to receive a bed of $1\frac{3}{4}$″ of concrete. Angle iron carriers shall support the treads and risers and shall be riveted to the strings.

Contractor shall furnish and erect, with all necessary hangers, the steel beams to support the intermediate stair landings. The beams for landings at floors will be supplied under another contract.

The newel posts may be made of cast iron, pressed steel, built-up steel parts, or extruded bronze. When cast-iron newels are used with rolled channels, they are blocked around the landing header, as shown in Fig. 490, and the string is bolted to the header beams through the newel.* Steel sub-tread stairs are generally made with closed strings which obviously necessitate a deep enough string construction to enclose the treads and risers.

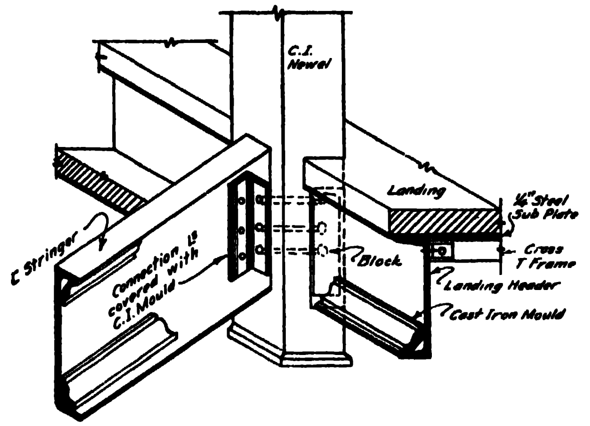


Fig. 490. Rolled Channel Stair Strings

The treads (and in some types, the risers) may be made of slate, marble,† concrete, mastic or a composition. Figure 491 shows some of the patented types of such construction. For slate or marble treads, the nosing is made by allowing the material to project, and the treads are set in plaster of Paris or elastic cement. For the other types, the steel sub-tread is turned up to make a form. The latter allows the use of safety treads.

Illustrative Prob. 307a. Design the steel sub-tread and riser stair construction for the details shown in Fig. 492, if

* Note in Fig. 490 how cast-iron moulds are used to cover the connections and the channel projections.

† One objection to slate and marble is that they disintegrate quickly when attacked by severe heat. The treads break during a hot fire and may cause accidents to firemen.

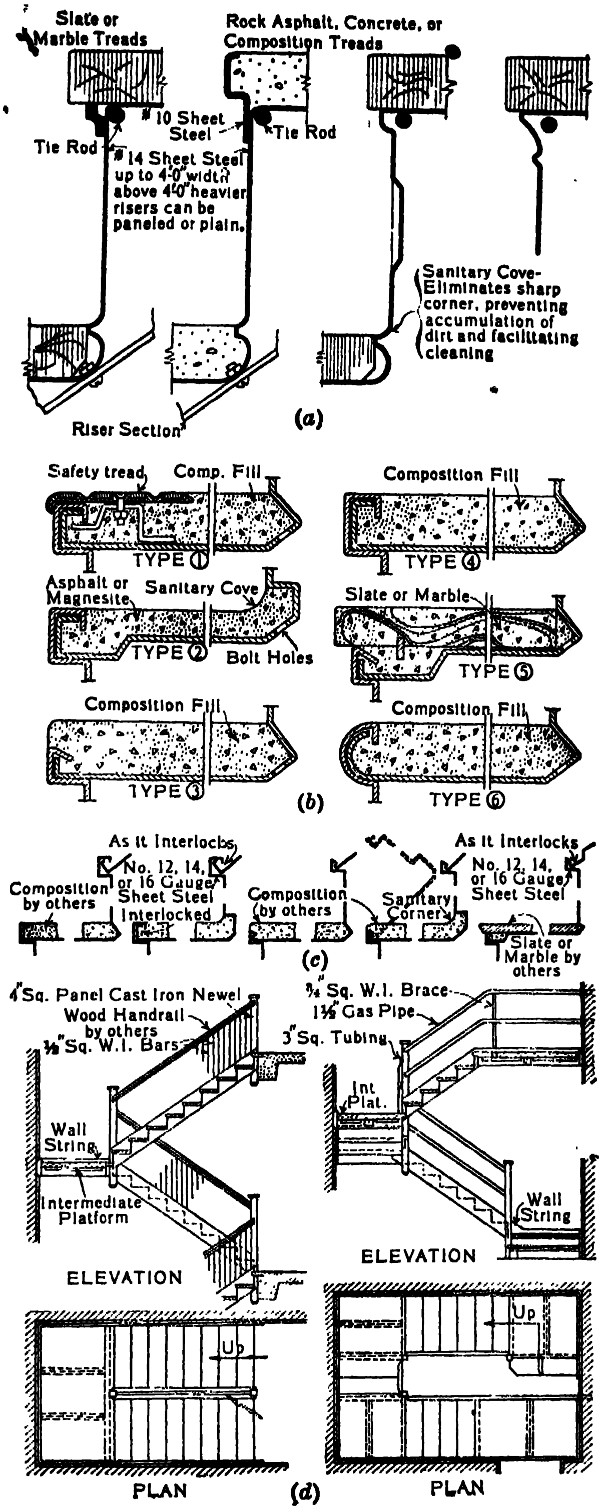


FIG. 491. STEEL SUB-TREAD AND RISER STAIRS

(*a*) Sexauer & Lemke, Inc.
(*b*) Bois Patent Interlocking
(*c*) Riester & Thesmacher Co.
(*d*) Woodbridge Ornamental Iron Co.

the risers are 7½" and the treads are 10". Width of stairway 4'-0" and stair run = 10 treads and a 4'-0" landing. L.L. = 75#/□'. Tread and riser plates to be #12 gauge.

L.L.	= 75	#12 gauge plate O.K. for this load between risers.
1¼" Slate	= 18	
Plate	= 2	
T.L.	= 95#/□'	$10 + \frac{3}{4} + 1\frac{1}{4} = 12'' = 1'\text{-}0''$

From treads 95 × 1.0	= 95
1" Slate riser	
14 × 6¼ ÷ 12	= 8
Riser plate	= 2
T.L.	= 105#/ft. on plate and angle riser form.

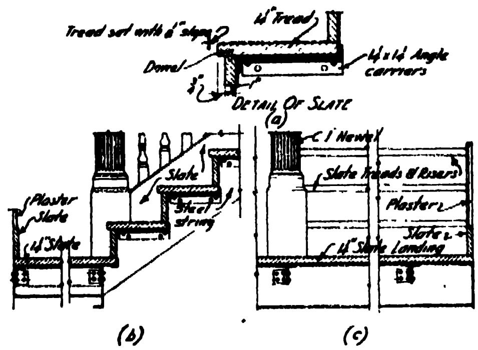


FIG. 492

L = 4'-0" between stringers

$M = 1.5 \times 105 \times (4)^2 = 2520''\#$

$\frac{I}{c}$ required $= \frac{2520}{16{,}000} = 0.16''^3$

I of $7\frac{1}{4} \times 0.1$ plate $= 3.17$

I of 2 ∟ $1\frac{1}{4} \times 1\frac{1}{4} \times \frac{1}{8} = 0.08$

$A \cdot d^2 = 0.30 \times 2 \times (3.28)^2 = 6.45$

Total $I = 9.70''^4$ $\qquad \frac{I}{c} = \frac{9.70}{3.63} = 2.77''^3$ O.K.

Use 10"-#12 ga. plates for treads.
7¼"-#12 ga. plates with 2-1¼ × 1¼ × ⅛ ∟ for risers.

By comparing the required and actual section moduli above, it may be seen that the material listed is sufficient for all ordinary cases without further investigation.

Stringers $L = 10 \times 10 = 100''$	= 8'-4"
Load per ft. = 105 × 2	= 210
Wt. of stringer	= 15
Slate facia	= 10
Railing	= 10
T.L.	= 245#/ft.

$M = 1.5 \times 245 \times (8.33)^2 = 25{,}600''\#$

$\frac{I}{c} = \frac{25{,}600}{16{,}000} = 1.6''^3$ required

Use 10" depth on a/c of framing in risers and treads (this depth should be established by a layout).

I of $10'' \times \frac{3}{16}''$ plate $= 16.86$

I of 2 ∟ $2 \times 2 \times \frac{3}{16} = 0.56$

$A \cdot d^2 = 2 \times 0.71 \times (4.43)^2 = 27.93$

Total $= 45.35''^4$ $\qquad \frac{I}{c} = \frac{45.35}{5.0} = 9.07''^3$

The stringers may be made of a 10 ⊏ 15.3 rolled section, a 10 × $\frac{3}{16}$ plate and 2-2 × $\frac{3}{16}$ angles built-up section, or 12 × $\frac{3}{16}$ pressed steel bent to 10″ ⊏ shapes, any of which has ample section modulus. In some cases, the landing header beam is omitted, especially when risers are introduced in the turn of the stairs. The stringers in such cases are bent to form the run of the stairs and then across to the wall on the far side of the landing. For stringers of rolled shapes or built-up sections, the two pieces are spliced by a plate. The group of rivets must be proportioned for the moment and shear at the point (Art. 256).

SPECIFICATION CLAUSE*

> In the construction of exterior stairs, landings, platforms and balconies, no rivet shall be less than ⅜″ diameter, and no bolt less than ½″ diameter.

In Fig. 492, the landing is formed by the use of a ⅛″ steel sub-plate, reinforced with tees about 2′-0″ o.c.

L.L.	= 75	
1¼″ Slate	= 18	113 × 2 = 226
Steel plate	= 5	Tee = 4
Susp. ceil.	= 15	T.L. = 230#/ft.
T.L.	= 113#/□′	$M = 1.5 \times 230 \times (4.0)^2 = 5530''\#$

$$\frac{I}{c} = \frac{5530}{16{,}000} = 0.35''^3 \qquad \text{Use } 2\tfrac{1}{2} \times 2\tfrac{1}{2} \times 4.9\# \text{ tees.}$$

Prob. 307b. Design the steel sub-tread and riser stair construction for the layout shown in Fig. 487 (*a*) if the L.L. = 100#/□′, and 1¼″ treads and 1″ marble risers are used. Width of stairs = 4′-0″. Risers 7½″, 14 treads at 10½″. Width of landing = 4′-6″. No header beam at landing to be furnished in structural contract. Use plate and angle stringers spliced and design splice. Make ½″ scale details of framing, and 3″ scale detail of typical tread.

308. Service Stairs.

Stairways in service portions and boiler rooms are generally made less expensive than public stairs, and often the treads are made with ⅜″ checkered plates, either with open risers or ones of checkered plates. The platforms may be made of the same material, supported by angle or tee frames. Rounded nosings are riveted to the strings. In some cases, the treads are made of a series of bars bolted together with pipe separators (gratings). Pipe railings may be used to reduce the expense. Otherwise, the construction is similar to that discussed in Arts. 306 and 307.

309. Emergency and Special Exits.

Although interior stairways, completely enclosed by fire-resisting partitions, are one means of egress, there are other methods of providing for exits. Since these are quite common in types of buildings in which the structural engineer may have a part of the responsibility of planning, he should be familiar with their characteristics. They may be classed as:

(1) horizontal exits,
(2) smokeproof towers, and
(3) outside exit stairways.

* From the Building Code of the National Board of Fire Underwriters, New York City.

A horizontal exit may be defined as one or more openings through or around a fire wall (or a bridge between two buildings), which afford a quick means of refuge on the other side, in case of fire. Such exits should be provided for floor areas exceeding 2000□′. The planning then resolves itself into the provision of fire walls and fire doors.

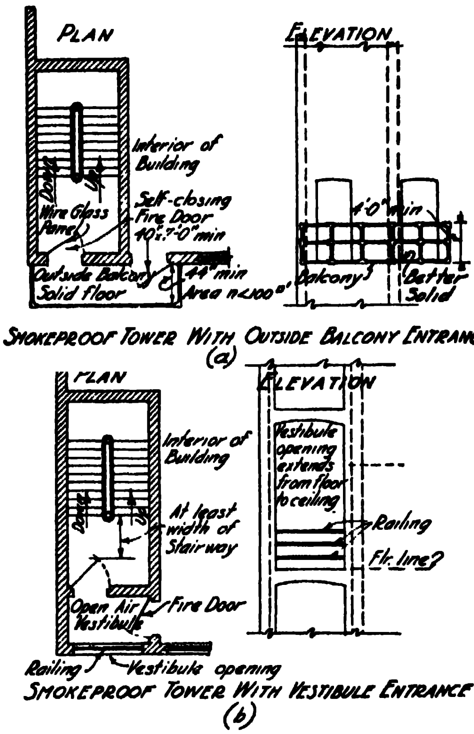


Fig. 493*

A smokeproof tower, illustrated in Fig. 493, is similar to an interior stairway, and extends from the grade to above the roof to form a bulkhead. It may open on to balconies of steel and masonry with solid floors, as in (*a*), or on to a vestibule within the main building lines but which is open on one side, as in (*b*). A detail similar to (*b*) may be used with entrances from two buildings, one at either side. Windows may be provided in the exterior walls if properly protected.

Outside exit stairways consist of balconies connected by stair flights all outside of the building line, made of incombustible materials. The stair flights may be either parallel or at right angles to the building, as shown in Fig. 494. The former has the disadvantage that the windows face the stairway, and in general, such stairs should not be in front of or over windows. It may be noted that the construction of these stairways is similar to that of service stairs (Art. 308) with open risers. The outside loads are carried by posts down to small footings, and the construction is not built on brackets extending from the

building, as is typical of some "fire escape" construction. A "balanced flight"* may be used at the bottom for buildings of moderate height. A disadvantage of an outside exit stairway is obviously the possible slipperiness due to ice, although a roof over it will help to eliminate this. Figure 495 shows a typical shop drawing. Such a stairway is of course not comparable with a smokeproof tower, from the standpoints of both efficiency and appearance.

FIG. 494†

"The ordinary so-called 'fire-escapes,' consisting of steel-framed balconies attached to a wall and connected by narrow steel ladders or steps leading from openings in the floors of the balconies, are considered very inefficient and unsafe means of exit. If any considerable number of people attempt to use such an exit, in time of fire panic, it quickly becomes so congested that travel is very much impeded or entirely blocked. If fire occurs on the floor below that from which people are endeavoring to escape, and the windows facing such exit are not protected by wired glass, the fire escape is worthless; and even with wired glass the exit is of doubtful value because of the intense heat which radiates through the windows. Such means of exit should never be permitted except upon existing buildings, where the number of people to be accommodated by them is small, and where structural conditions are such that it is impossible to secure anything better. They are not recognized as a required means of exit in this Code."†

310. Circular Stairs.

Occasionally where room for secondary stairways is limited, as in library stack rooms, boiler rooms, and so on, circular stairs are used. Figure 496 shows the general characteristics. The treads are cantilevered from a central mast, commonly a 4″ wrought-iron pipe column with a flange at the bottom. Such stairs are manufactured by special companies and are available in 3′-6″, 4′-0″, 4′-6″, 5′-0″, 5′-6″, 6′-0″ and 7′-0″ diameters, although the 4′-6″ and 5′-0″ stairs are the most common. They are made with either 12 or 16 treads to the circle. To provide sufficient headroom under the top platform, the risers should not be less than $8\frac{1}{4}$″. Hence by calculating the number of risers required, the relative positions of the starting and landing points may be determined.

The two important points which interest the structural engineer are the load concentrated at the foot of the supporting post and the framing around the stair opening. Circular stairs weigh about 50# per foot of height on the average. With this value, the approximate weight of the stairs may be calculated. It is doubtful if it is necessary to figure much live load on a stairway of this kind. Not more than one or two persons would be using such a stair at one time except in cases of extreme emergency and then the factor of safety would be ample protection. It is recommended that a total live load of 500# be added to the dead load. The local building regulations must, in any event, be satisfied.

* This tips down as a load is thrown out upon the extending arm, similar to a "teeter board." This prevents entering the stairway from the ground.

† Excerpted from the "Building Code" of the National Board of Fire Underwriters, New York.

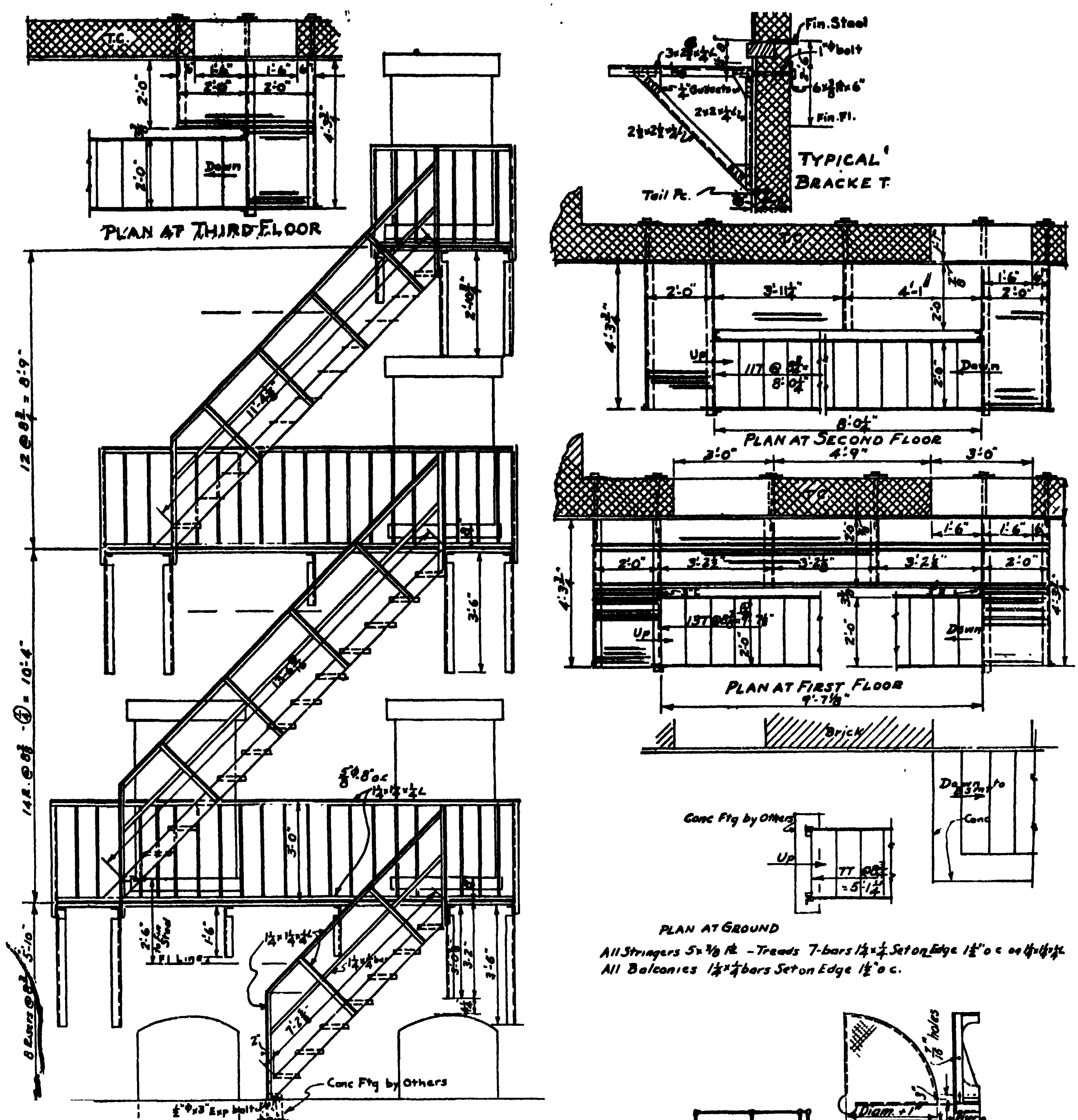


Fig. 495

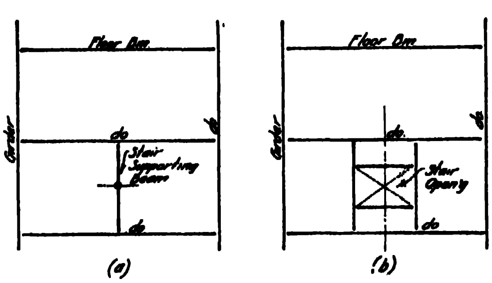


Fig. 497

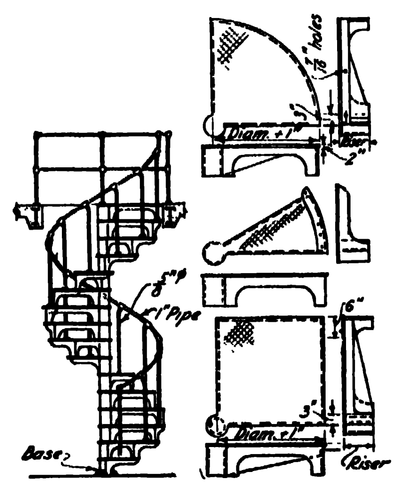


Fig. 496

The openings for circular stairs may be either square or round. The side of the square, or the diameter of the circle, should be made 2″ greater than the nominal diameter of the stairs. The platforms are made either square or circular, and the side of the square or the radius of the circle is made one-half of the dimension fixing the size of the opening. In a steel-framed floor, a supporting beam is generally used under the mast, as shown in Fig. 497 (*a*) and the mast embedded in concrete. Header beams are usually provided around the floor openings above, as in (*b*).

FIG. 498*

311. Ladders.

In engine rooms, boiler rooms, elevator penthouses, and so on, steel ladders are often provided. These are made up of strings of 2″ × $\frac{3}{8}$″ bars, 14″ apart, with $\frac{3}{4}$″ ϕ rungs spaced 12″ o.c., usually. Small angle clips may be used at the bottom, expansion bolted to the masonry or concrete, as the case may be. Similar clips may be used at the top, or half turns may be made in the bars to form elbows, so that the ladder is set a few inches away from the face of a wall. The clips or elbows may be expansion bolted in a manner similar to the bases.

312. Escalators.*

For subway, elevated and railroad terminals, transportation of employees in industrial buildings, to balconies of theatres, and in department stores, there is a tendency toward the more frequent installation of escalators as a means of travel. This is especially true for industrial buildings of 3 or 4 stories where it is necessary to move large numbers of employees at fixed times, and results in greater efficiency of the employees. For more than 4 stories, elevators are more economical in the usual case for all traffic. Of course elevators are necessary for all buildings to handle the ordinary light travel. Figure 498* shows a typical installation of an escalator. An escalator consists essentially of the following parts: (1) the running gear, or moving steps, which conveys the passengers, (2) the track system, which supports the running gear, (3) the balustrading provided with hand rails moving at the same speed as the running gear, (4) the drive, or machinery necessary to operate the running gear and hand rails, (5) the safety features designed to stop the escalator automatically in cases of emergency, and (6) the steel structure, or skeleton of the entire escalator body. Two types are common, — the step (Fig. 498), and the cleat. The first starts at the bottom with a moving platform which breaks into steps, and at the top it again flattens out into a moving platform. This mechanism is mounted on wheels which run on a curved inclined track. The second type consists of hardwood cleats located in long ridges and grooves, which are bolted to a steel bushed chain which in turn passes over sprocket wheels. Either type may be made to run up or down, or duplex, or with reversible switches. For rises of 25′-0″ or less, ascending service only is common for terminals, while for rises exceeding 25′-0″ both ascending and descending service is usually provided. The usual incline is 30° and the common travel is 90 ft. per minute on the incline. Large numbers of persons may be handled with such service. The following tabulation is of value in establishing requirements:

2′-0″ wide, single file	4000 persons per hour
3′-0″ " " "	6000 " " "
4′-0″ " double file	8000 " " "

The structural engineer is primarily interested in the arrangement of the beams which surround the openings in the floors involved, and the loads which are brought upon the beams. The typical layout is furnished by some company specializing in such work, from which the engineer spaces his beams and computes their sizes in the usual way to carry the loads given. Figure 499* shows a typical layout of this kind, as supplied by an escalator company. The loads given are such that if a working stress of 12,000#/□″ is used in the design of the beams, no allowance need be made for vibrations, impact, etc. The loads include the complete escalator with live load, but without drive and drive chamber. The loads should be figured as uniformly distributed on the cross beams. If the escalator is to support any housing or any additional load, the manufacturer should be consulted. It should be noted that the loads are dependent upon the vertical rise of the escalator (see tabulations in Fig. 499).

The structural engineer is occasionally called upon to make a study of the requirements for planning a layout. The following* is a typical set of guide notes:

* The notes, figures and excerpts upon which this article is based were furnished through the courtesy of the general offices of the Otis Elevator Co., New York City.

Escalators, where they have been installed in industrial buildings for handling employees, have proven* to be both profitable and practical. The principal reason for the results obtained is due to the increased efficiency of the employees, on account of the elimination of stair climbing. While it is almost impossible to figure the actual saving, a careful study has convinced the owners that a vast sum of money has been saved by conserving the energy of the employees. Furnishing a quick and efficient means for the transportation of the employees to the upper floors also results in attracting and holding a better class of labor. The question as to whether escalators are preferable to elevators for such service can only be determined after a careful analysis is made in each case, taking into account the elevator equipment that will be required in addition to the escalators and the possibility of their being available for use at the time employees are being transferred.

In general, for conditions where a large number of employees are to be handled in a short period of time and where about three to four stories are involved, it is quite probable that escalators will be the most economical both as to first cost and cost of maintenance and operation. For high buildings, elevators in the majority of cases are more suitable, for in such cases several elevators are required for service during the day, and they may be used with but slight additional equipment for handling employees. As escalators run at 90 feet per minute on the incline, which is equivalent to approximately 45 feet per minute vertically, it can readily be seen that the time required to reach the upper floors at that speed on escalators would be more than could be allowed.

There is a great variety of conditions involved in connection with the use of escalators for handling employees. In the case of some department stores the escalator equipment that is used through the day by the public is called into service to take the employees up to their respective departments in the morning and out at night, and this could not be the case in industrial buildings, as the general travel throughout the day is comparatively light, and is more economically taken care of by the auxiliary elevator equipment.

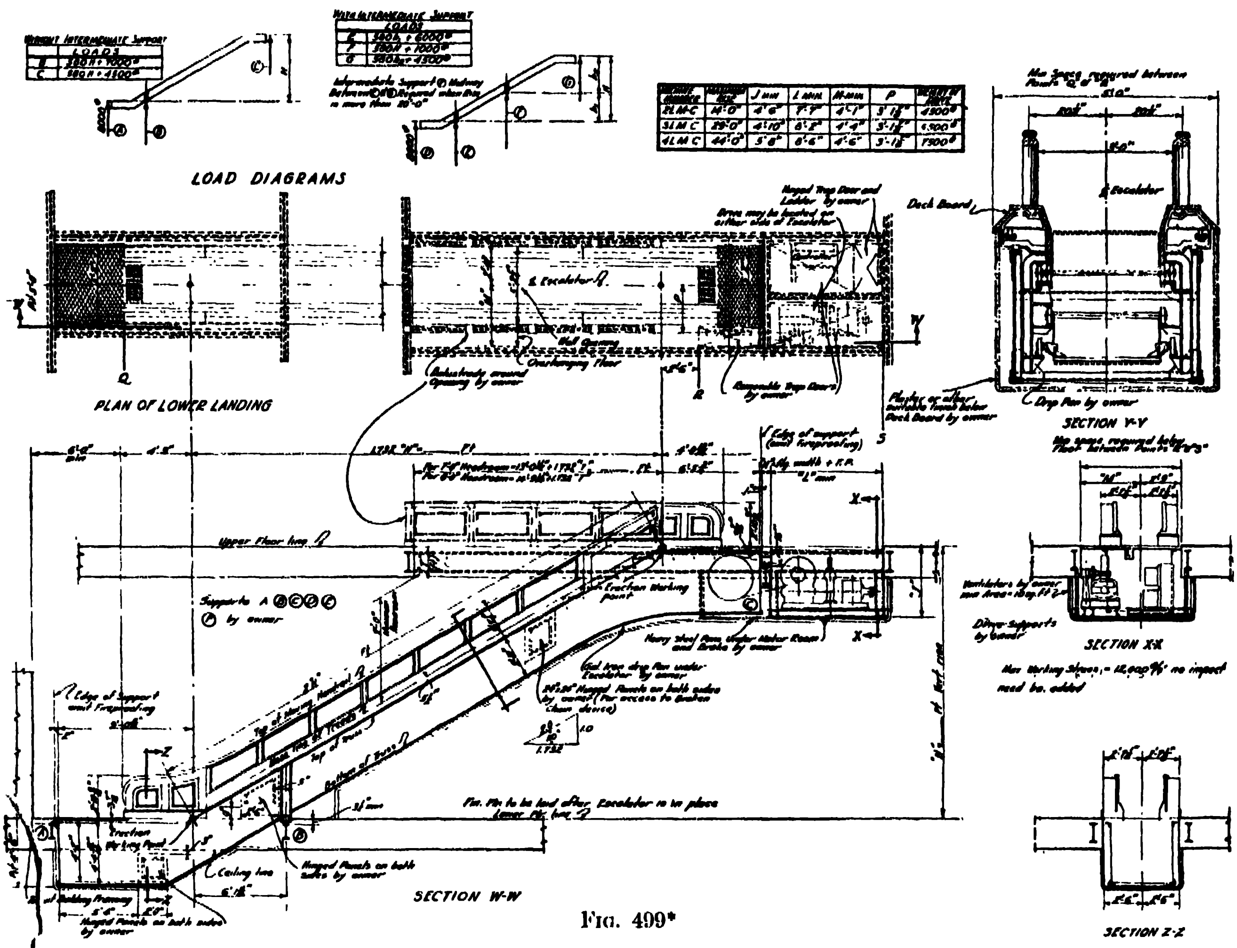


Fig. 499*

There are some cases where escalators are installed to serve the employees' lunch and rest rooms. In some such cases an escalator is installed for that purpose only, as even if there were escalators for handling the public they could not be used by the employees during the busy part of the day, while there are some other cases where it is possible to use the general escalators for that purpose.

From the above it will be noted that there are innumerable and widely varying conditions involved in almost every case; therefore, no definite general recommendations or comparisons of escalators and elevators as to space conditions, cost, etc., can be made. It is necessary to know the number of persons to be handled, the time allowed, the number of floors, the number of persons on each floor, the height of floors, suggested and allowable location of escalators, full data concerning the elevator equipment to be used in addition to the escalators before it is possible to determine the number and type of escalators.

* Courtesy of Otis Elevator Co.

CHAPTER 27

WIND BRACING*

313. General.

When high, narrow buildings, 12 stories or more in height, are planned, special provisions must usually be made to supply resistance to the action of the wind. Such structures are of the skeleton frame type with curtain walls, for economic reasons. This type of frame may lack sufficient rigidity to resist wind pressure safely unless this is specifically provided for.† In buildings less than 100'-0" high, the lateral resistance of the girder connections, aided by the stiffness of the walls, is usually sufficient. When the width of a structure is small compared with its height, however, some form of bracing is used. Although building codes vary in their requirements and some give no limitations, a general, guiding rule is that **if the width is less than one-half the height, wind bracing should be employed** (Fig. 500).

Wind bracing is theoretically not necessary for sheltered places, but it is wise to provide for the **wind pressure based on the area offered by the side of the building.** Fire or the removal of an adjacent structure may expose the whole of a wall surface to the action of the wind. Exterior walls, interior walls and partitions, and the fireproofing of the frame give stiffness to a building, but the amount is indefinite and practically impossible to calculate, as it depends upon the height of the structure, the number of partitions and the relative locations of openings in them. While this stiffness is effective, it is inadequate and it should be neglected, and **wind bracing should be designed to resist the full wind pressure.**

314. Routing the Stress.

The wind pressure must ultimately be resisted by the foundations, but the method to transmit this thrust may be varied. The curtain walls and windows are strong enough to carry the local pressures to the spandrel beams. The floors tend to act as horizontal girders and have ample strength to transfer pressures to the frame. Any small, lateral movement at local points in beams and columns will be absorbed by the connecting beams. The spandrel beams usually have considerable excess resistance to shear, so that pressure may be carried in a horizontal, longitudinal direction.‡ **The principal problem then becomes that of determining how the pressure should be transferred from the frame to the foundations.**

Theoretically, the bracing should be placed where it will be the most effective and where it will interfere the least with the architecture. Often a study of the floor plans will be helpful. One method would be to carry the wind pressure along the sides of the building by the spandrels to the ends, as in Fig. 501 (*a*), and apply it at the ends parallel to the floor beams. It is generally more feasible to take advantage of all of the interior framing and to carry the loads through each lateral row of columns as in (*b*). Less metal is required in any individual member, and the total amount required is generally less. This method is then generally more economical, in spite of the added cost of fabrication. Hence **each bent of the structure is usually designed to resist its share of the wind pressure.**§ A structure is seldom braced in its longitudinal direction, particularly when the length is 100'-0" or more. Bracing in horizontal planes is generally not necessary, as the floors are amply stiff.‖

The action of an individual bent may be likened to that of a cantilever plate girder with the fixed end at the foundations. The columns and beams in the outside walls correspond to the flanges, and a bent of vertical framing (or an end wall) corresponds to the web. Instead of having a solid web plate, however, it consists of a series of openings corresponding to the stories of the building. The amounts of the shears are the same, but the character of the secondary stresses developed by them depends upon the relative positions of the openings. The object of the wind bracing is to transmit the

* The discussion given here relates principally to wind bracing for high, narrow buildings. For that relating to mill buildings and braced bents, see Part VI.

† Some high structures have been found which had no wind bracing that were as much as 6" out of plumb.

‡ If insufficient resistance is found, bracing in the end panels of the side walls may be introduced.

§ In special cases, heavy girder connections may interfere with the architecture. The first method described may have to be used in such cases. A combination of the two methods may be used also, in which the pressure above a certain floor is carried by the interior frame, and the pressure below that floor by a frame in the end walls. This is not advisable for ordinary cases.

‖ Where this is necessary for special cases, bracing may be provided between two selected sets of columns in the longitudinal direction.

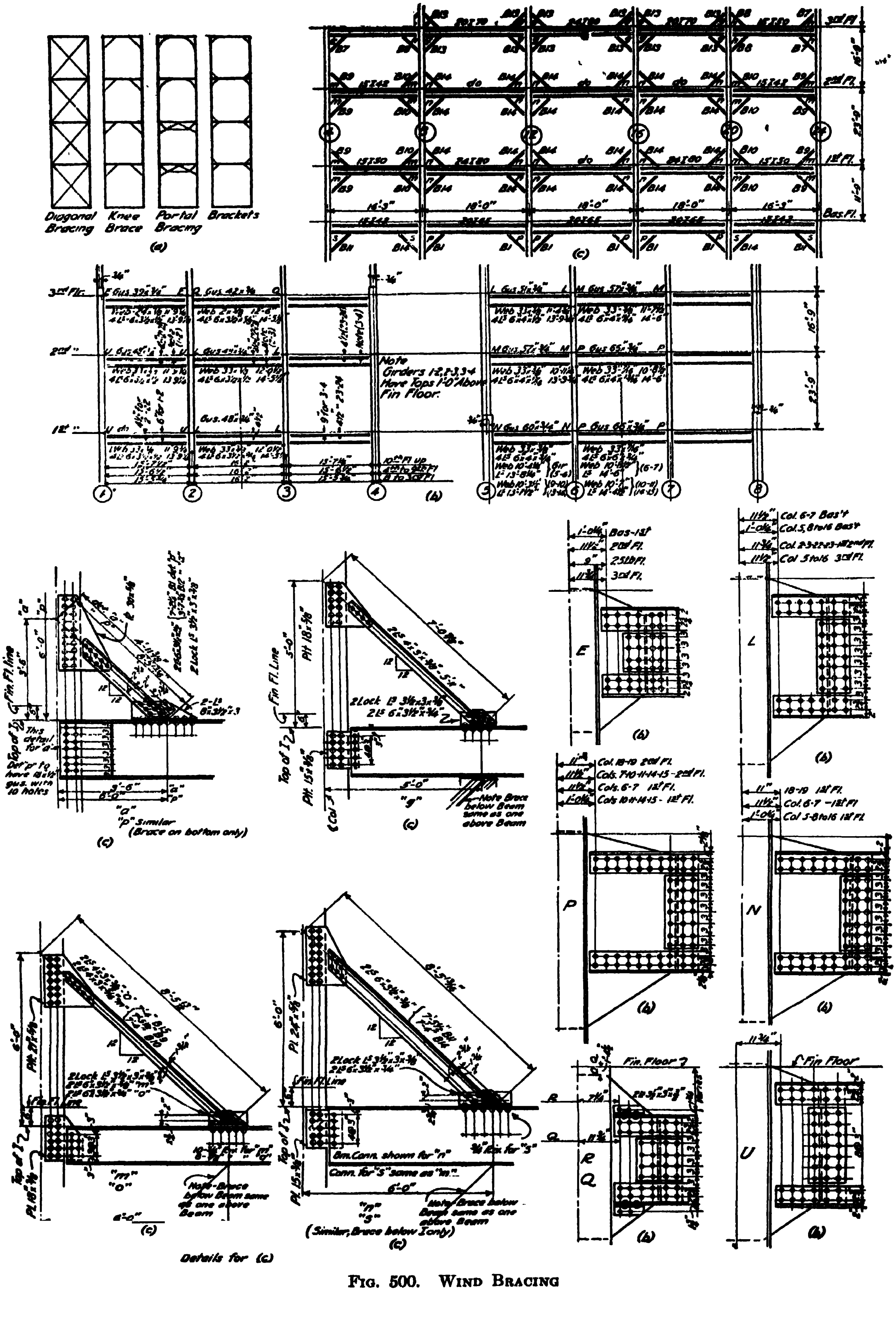


FIG. 500. WIND BRACING

shear: The shears at each level of bracing are determined, and these are used to analyze the bending moments and shears induced in the individual members.

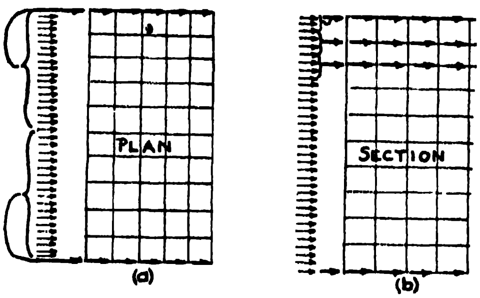


Fig. 501

315. Types of Bracing.

Various types of bracing have been used to provide resistance against wind pressure. These may be grouped as:

(1) portal,
(2) sway-rod, or diagonal,
(3) knee bracing, and
(4) brackets.

Portal bracing may be either latticed, as in Fig. 502 (*b*), or built up of plates and angles as in (*a*). It is cumbersome and expensive, and is seldom employed in modern work. It is advantageous where there are large spaces between the columns. A portal is in reality a steel arch, and in some cases it may be adapted to the architectural treatment and also to carry the floor loads. Considerable depth is required for stiffness, and when made deep, the stresses are relatively quite small. With minimum sizes of shapes, no economy results.

Sway-rod, or diagonal bracing, as illustrated in Fig. 502 (*c*), was used principally in the early methods of the wind bracing of buildings. While it is the most direct and efficient method, it interferes with openings and cuts up the partitions. It is the only system not causing bending in the girders and columns, if all the axial lines of the members intersect. Knee bracing, as shown in Fig. 502 (*d*), is sometimes used, particularly when wind bracing is to be employed in the outside walls. Braces above and below the girders may be used, although braces under the girders are more common, as the gain with top knee-braces is small compared with the increased amount of metal and fabrication. An objection to this type of bracing is that the girders and columns are subjected to large bending stresses with the incident eccentricity caused by flexure.

Wind bracing of the bracket type may consist of extra heavy beam connections for light framing, as in Fig. 502 (*e*), or of gusset plates, as in (*f*). **Bracket connections are the common type used in wind bracing of modern buildings.*** The principal advantage is that such bracing may be made to conform with the architectural treatment most readily. More rigid connections are required, a great amount of field riveting is necessary, and considerable bending is induced in the girders and columns, but in spite of such disadvantages, such connections are generally considered to be the best type.

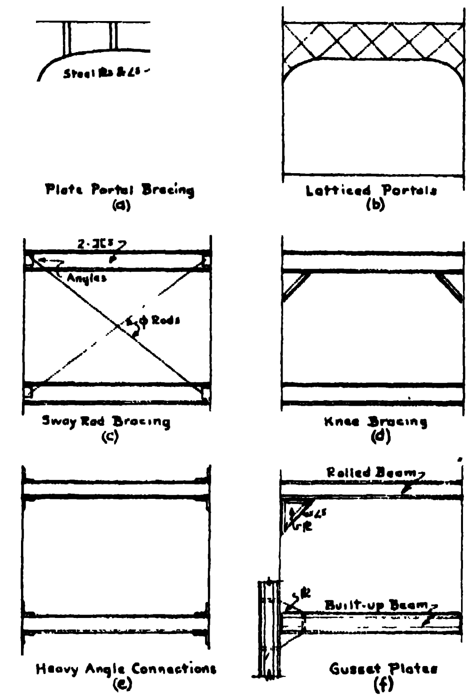


Fig. 502

316. General Requirements for Bracing.

Irrespective of the type of wind bracing used, its arrangement should be made symmetrical about the axis of each face of the structure. If the building is irregular in shape, this is not always possible, and there would be a tendency to twist the building about a vertical axis. The best protection against such action is to use deep spandrel girders which can absorb the twist.

One of the general requirements of the bracing is to prevent the distortion of the frame. The main horizontal and vertical members must be sufficient to resist the bending produced by the wind action as well as that caused by the direct loads. Their connections must be strong enough to resist the vertical load as well as the sidewise thrust.

In special cases where elevators and stairs are next to a wall throughout the typical stories, a plate

* Further discussion of other types of wind bracing will not be given here. For special instances, reference may be had to Kidder's "Architects' and Builders' Pocketbook,"—John Wiley and Sons, Inc., Publishers.

riveted on the top flanges of the beams at each floor from the spandrels to the interior framing may be made to serve in causing these cross beams to transfer the wind pressure to the interior bracing system. The connections must be made strong enough to resist the thrust and the cross beams should be checked up, acting as struts.

The resistance to the wind should preferably be made by a large number of connections rather than by a few selected, heavy joints. The use of double beams is advantageous, but they are usually unsightly if exposed, add considerably to the cost, and require a larger amount of fireproofing.

317. Wind Pressure.

The general theory of wind pressure is discussed in Art. 160. No special considerations are necessary as related to wind bracing. Although the actual wind pressure may be greater near the top of a building than near the base, and it may be inclined, due to the contour of the ground or due to obstructions, the pressure for design purposes is **assumed to be horizontal and to be uniformly distributed** over the windward surface of a building, occurring in any direction.

The intensity of pressure is specified as different amounts in various building ordinances. Some codes require 30#/□′ over the exposed surface from the ground to the roof, while others specify 20#/□′, and still others a variable pressure according to the number of stories in the building. In the absence of other regulations, **a unit pressure of 25#/□′ is recommended,** unless a building is in a known area of atmospheric disturbance, when 50#/□′ is advised.* While higher pressures may be recorded, these will occur only once or twice in the average life of a building, and the factor of safety is ample for them.

SPECIFICATION CLAUSES †

All buildings or parts of buildings in which the height is more than three times the minimum horizontal dimension shall be designed to resist a horizontal wind pressure in any direction of 20 lbs. for every square foot of exposed surface. Wind bracing shall be provided by making the connection joint between girders and columns sufficient for the vertical load as well as the bending due to side pressure; or diagonal bracing shall be placed between columns, proportioned to transfer the shear of the side pressure to the footings. All details shall be designed to carry the stresses in the main members.

The overturning moment due to wind pressure shall not exceed 50 per cent of the moment of stability of the structure, unless the structure is securely anchored to the foundation. The anchors shall be of sufficient strength to safely carry the excess overturning moment, without exceeding the allowable unit stresses given in this Code.

When the stress due to the wind in any member or connection amounts to less than 50 per cent of the total live and dead loads, it may be neglected. When the stress due to the wind exceeds 50 per cent of the stress due to the combined live and dead loads, all these stresses shall be added together and the allowable unit stress for the total may be taken at 50 per cent in excess of the values stated in Sections 65 and 66. In no case shall the section be less than required if wind forces be neglected.

In proceeding with the design of wind bracing, it is necessary to calculate the **wind panel loads** at each floor. This is of course the result of the products of the unit pressure, the story height and the spacing of the wall columns. In determining the loads at the roof lines, any parapet or cornice should be included. In making further computations, the successive summation of the pressures for each story from the sidewalk to the top of the building must be known. This is based on the assumption that the strength of the frame above any given floor is sufficient to transfer such thrusts. These loads are calculated for each floor down to the roof of an adjoining building or wing, or to the plane of the bracing where it will be designed to carry the bottom pressure through all the tiers below. The latter is one of the most important points, and one which is difficult to ascertain, namely, to **decide how far down from the top it is safe to assume that the building is rigid enough to resist the wind pressure itself.** When there is any reasonable doubt it is always wise to check all floor lines.

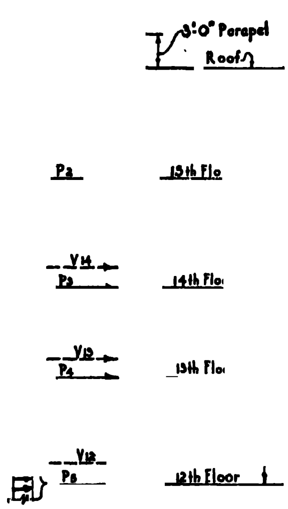


FIG. 503

Illustrative Prob. 317a. Determine the wind panel loads in Fig. 503, and the total shears at each story, if the spacing of wall columns is 20′-0″.

$P_1 = (3 + 9) \times 20 \times 25\#/\square' = 6000\#, \; V_{15} = 6{,}000$
$P_2 = (9 + 8) \times 20 \times 25 = 8500, \; V_{14} = 14{,}500$
$P_3 = (8 + 6) \times 20 \times 25 = 7000, \; V_{13} = 21{,}500$
$P_4 = (6 + 6) \times 20 \times 25 = 6000, \; V_{12} = 27{,}500$, etc.

Another feature which should be investigated is the stability of the structure, regardless of the interior bracing. For safety, the overturning moment due to the wind should not be more than 75% of the moment of resistance. If the bending moment should happen to exceed the moment of stability,

* Recent studies now in progress (1927) following the Florida hurricane, are indicating that very high unit wind pressures, with an impact effect, existed.

† From the Building Code of the National Board of Fire Underwriters, New York City.

upon an unloaded building, the columns be anchored for the uplift generated. The total uplift on the corner columns may exceed the dead and live load, but this is not serious if the uplift is transferred to the side walls by girders.

Prob. 317b. Determine the typical wind panel loads and total shears for the upper ten stories of a building with wall columns 18′-0″ on centers, 2′-0″ parapet, 16′-0″ top story height, and 14′-0″ typical story heights. Wind pressure = 30#/□′.

318. Methods of Analysis.

There are a number of different methods of analyzing the stresses induced in the girders, columns and bracing by the wind pressure. There is no theory which is exact, although the "slope-deflection" method * is the nearest approach. This is too involved and laborious for ordinary commercial use. Practical methods require assumptions which, incidentally, are not consistent with each other, but which are necessary in order to make solutions reasonably simple. The most important of these are:

(*a*) that all joints are perfectly rigid,
(*b*) that the points of contraflexure in each column are at one-half the height,
(*c*) that all columns in any given story have the same sectional area and the same moment of inertia.

The following practical methods of analysis are used, and may be classified by name as: (1) cantilever, (2) equal shears, and (3) balanced bays.

The first method conforms the nearest to the discussion in Art. 314, and probably is most in accordance with the true distribution of stresses. However, it is more complicated than the others. In the method of equal shears, (2), it is assumed that the horizontal shear at any plane is equally distributed among the columns cut by that plane. This infers that the bending moments in all the columns in one tier are equal. The third method is a variation of the second in that it is assumed that the **total horizontal shear at any plane is divided by the number of bays** (spaces between the columns). This means that the shears and bending moments in the outside columns are respectively one-half of those in the interior columns. It also means that the bending moments (due to wind) are alike for all girders on the same floor in any transverse bent, and that the points of contraflexure in all the girders are at their mid-spans. The direct axial stresses generated in the columns by the overturning moment of the wind all occur in the outside columns, as those in the interior columns neutralize each other. **The above conditions of course hold true only for an equal transverse spacing of columns.** With an unequal spacing of the columns across a building, the wind load taken by each bay should be assumed in proportion to the ratio of the span to the width of the bent. This will bring all the direct, axial stresses to the outside columns (as for equal spacing), — those in the interior columns neutralizing each other. The horizontal shear in the columns should be assigned in proportion to their relative moments of inertia. This will mean that the respective bending moments in the girders will vary according to the spans, although the points of contraflexure will be at each mid-span point.

The choice of the method to use in determining wind stresses may vary according to the particular building to be analyzed, and in unusual cases, combined or even special methods may be used, involving very careful study, such as the Woolworth Building in New York City. The method of balanced bays is simplest to apply, and because of the equal bending moments in girders on one floor of equal bays, typical details and fabrication result. It is also a conservative method. In general, for the above named reasons, **the method of balanced bays is recommended,** and it will be the only method analyzed here in any detail. †

319. General Theory.

Before the application to a large frame can be well understood, and the discussion previously given really comprehended, the application to simpler frames should be studied. In Fig. 504 (*a*), if the frame is hinged as shown, it would collapse when subjected to a pressure, W, as indicated by the dotted lines. If the ends were fixed as in (*b*), the distortion would be similar to that shown in an exaggerated way, with points of contraflexure occurring part way up the posts (assumed at one-half the height). It is this tendency toward distortion which causes stresses in the beams and columns in a real structure. Theoretically, no bending occurs at the points of contraflexure and hinges could be introduced at these points. Figure 504 (*c*) shows a hypothetical illustration of this. Taking out any part as a free body, such as ABC, in Fig. 504 (*d*), the following relations apply:

$$\text{Bending moment at } B \text{ in vertical member} = \frac{W}{2} \times \frac{h}{2} = \frac{W \cdot h}{4}$$

$$B \text{ in horizontal member} \quad V \times \frac{L}{2}.$$

$$\Sigma M_B = -V \times \frac{L}{2} + \frac{W}{2} \times \frac{h}{2} = 0, \text{ or } V = \frac{W \cdot h}{2\,L}.$$

* Refer to Bulletin No. 80, "Wind Stresses in the Steel Frames of Office Buildings," by W. M. Wilson and G. A. Maney, Engineering Experiment Station, University of Illinois, also to Volume 53, No. 12, Engineering and Contracting, p. 314.

† Study of other methods may be had by reference to Kidder's "Architects' and Builders' Pocket Book," John Wiley & Sons, Inc., Publishers, and to an article in the Journal of the Boston Society of Civil Engineers, May, 1926, entitled "Wind Bracing in Industrial and Many-Storied Buildings," by Mr. Robins Fleming, Structural Engineer, American Bridge Co., New York City.

Substituting, for V, the bending moment at B in the horizontal member $\frac{W \cdot h}{2L} \times \frac{L}{2} = \frac{W \cdot h}{4}$.

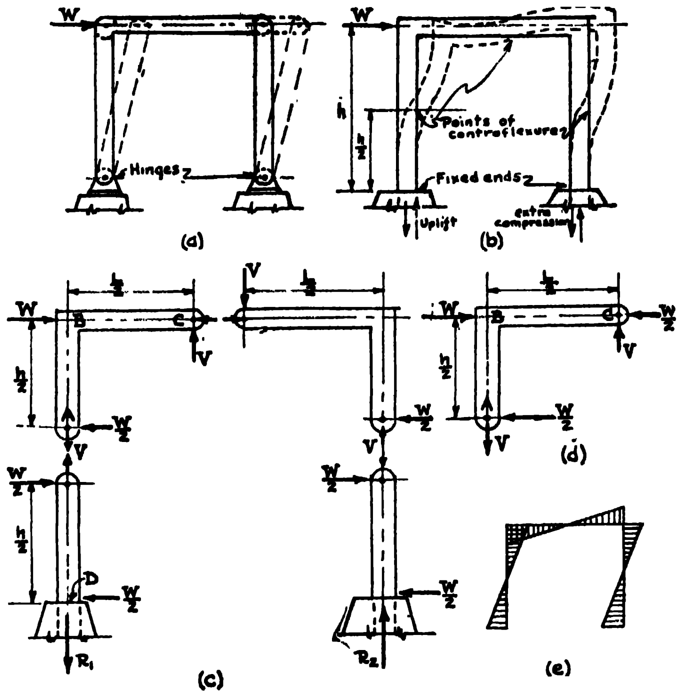


FIG. 504

The bending moment in the post at D (considering the lower portion in (c) to be a free body) $= \frac{W}{2} \times \frac{h}{2} = \frac{W \cdot h}{4}$.

It will be noted that all the bending moments are equal. Fig. 504 (e) shows a moment diagram. The direct stresses may also be computed. Thus by $\Sigma V = 0$ (for the lower portion of the post as a free body),

$\therefore R_1 = V = \frac{W \cdot h}{2L}$ tension, and R_2 $\frac{W \cdot h}{2L}$ compression

The same reasoning may be extended to two panels, as shown in Fig. 505 (a). This may be considered as two separate panels, each carrying one-half of W. The bending moment at the base of column ① is $\frac{W}{4} \times \frac{h}{2} = \frac{W \cdot h}{8}$. The bending at the base of column ② is $\times \frac{h}{2} = \frac{W \cdot h}{4}$ The bending in all cases, including both girders, will be $\frac{W \cdot h}{8}$ except at the base and top of column ②. Column ② is common to two panels, hence the bending is $2 \times \frac{W \cdot h}{8} = \frac{W \cdot h}{4}$. The axial stresses in column ② tend to neutralize each other, and if $L_1 = L_2$ the result is 0.

In a similar manner, the reasoning may be extended to any number of panels, as in Fig. 505 (b), although the usual limit of considerations for wind bracing is 4 or 5 columns in the plane of the bracing. W divided by the number of panels equals the force on each interior column. The force on an exterior column is one-half that on an interior column. Thus for the 5 bays in Fig. 505 (b),

Bending moment at base and top of columns ① and ⑥ $= \frac{W \cdot h}{20}$. For columns ②, ③, ④, and ⑤,

$$M = \frac{W \cdot h}{10}. \quad \text{For all girders, } M \quad \frac{W \cdot h}{20}.$$

When two tiers are considered, as in Fig. 506 (a), the lower story frame serves as the foundation for the upper story. If the frame were considered to be a cantilever beam with a solid web, as discussed in Art. 313, it would be possible to find the internal stresses at any point. When rectangles are cut out by the stories, however, conditions are altered. That which would correspond to the shear parallel to the base in the solid webbed beam (vertical shear in the beam action) is taken by the columns as shears at their points of contraflexure. That which would correspond to the

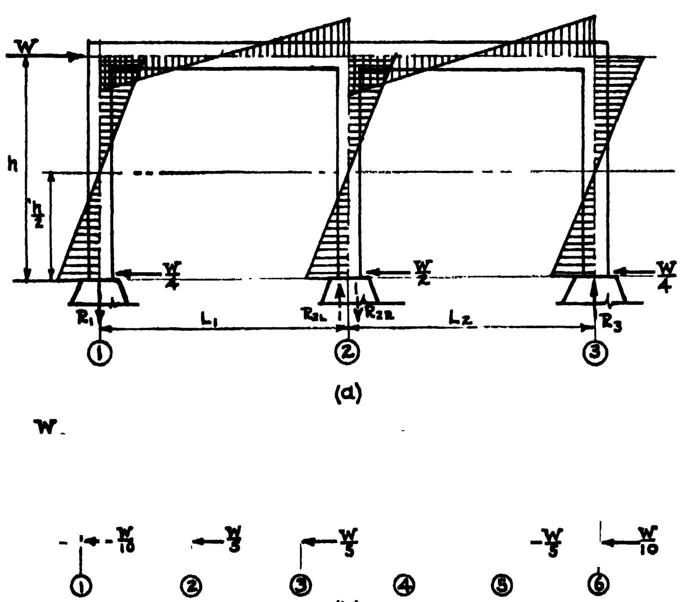


FIG. 505

shear parallel to the sides (horizontal shear in the beam action) is taken by the girders as shears at their points of contraflexure. These shears cause bending at the junctures of the girders and columns,

as explained before. The values which would correspond to the tensile and compressive stresses in the flanges of the cantilever beam are taken as direct loads by the outside columns (see Fig. 506 (*b*)).

Referring to Fig. 506 (*a*), the moments at the bases and tops of columns and in the girders may be calculated, using the same principles as before. Thus,

At point *A* in roof girder,

$$M_A = -V_R \times \frac{L}{2} = -\frac{W_R \cdot h_2}{6\,L} \times \frac{L}{2} = -\frac{W_R \cdot h_2}{12}.$$

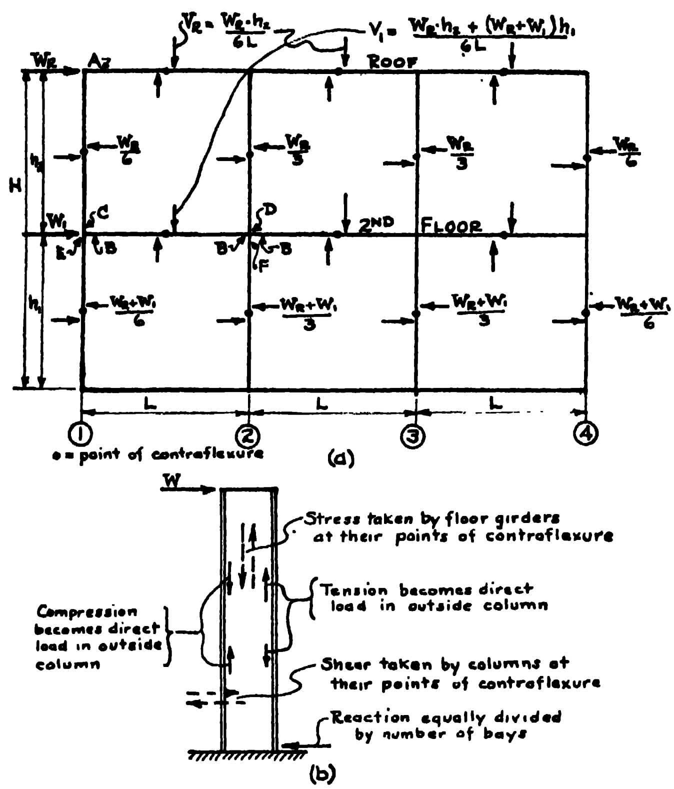


FIG. 506

By similar moments on the joints as free bodies,

At point *B* in 2nd floor girder,

$$M_B = -\frac{W_R \cdot h_2 + (W_R + W_1)h_1}{12}.$$

At point *C* in 2nd story outside column,

$$M_C = +\frac{W_R \cdot h_2}{12}.$$

At point *D* in 2nd story interior column,

$$M_D = +\frac{W_R \cdot h_2}{6}.$$

At point *E* in 1st story outside column,

$$M_E = +\frac{(W_R + W_1)h_1}{12}.$$

At point *F* in 1st story interior column,

$$M_F = +\frac{(W_R + W_1)h_1}{6}.$$

Each intersection of a column with a girder is held in equilibrium by the forces acting at the respective points of contraflexure. By the laws of equilibrium, the sum of the moments at any point in a member must equal zero. Thus for column ① in Fig. 506 (*a*), at the second floor, the algebraic sum of M at B, M at C, and M at $E = 0$. Using their respective values, the following equation results:

$$-\frac{W_R \cdot h_2 + (W_R + W_1)h_1}{12} + \frac{W_R \cdot h_2}{12} + \frac{(W_R + W_1)h_1}{12} = 0.$$

Stated in a converse manner, **the bending moment in the girders equals the sum of the moments in the outer columns above and below the floor.** Thus in Fig. 506 (*a*), $M_B = M_C + M_E$. Similarly, at any interior joint as well, the sum of the moments in the columns equals the sum of the moments in the girders. At column ②, second floor, in Fig. 506 (*a*), M_B (left) $+ M_B$ (right) $= M_D + M_F$. Using their respective values, the following equation results:

$$\frac{W_R \cdot h_2 + (W_R + W_1)h_1}{12} \times 2 = \frac{W_R \cdot h_2}{6} + \frac{(W_R + W_1)h_1}{6}.$$

Following the same procedure, the moments in the columns and girders may be obtained for any number of stories. Referring to Fig. 507, the wind panel loads are designated by W_R (roof), W_6 (sixth floor), W_5, etc. The total shears in any story equal the sum of all the loads applied at the floors above. These are designated by V_6 (sixth story), V_5 (fifth story), V_4, etc. Thus $V_3 = W_4 + W_5 + W_6 + W_R$. The total shear in any story divided by the number of panels equals the shear resisted by each interior column. One-half of this amount is resisted by each exterior column. From a study of the preceding cases, the following general rules and formulas may be given:

(1) **The bending moment at an interior column in any story is equal to the total shear in that story times the story height, divided by twice the number of panels in the transverse bent.**

Expressed as a formula,

$$M_{IC} = \frac{V \cdot h}{2\,n}, \text{ in which} \qquad (S\text{–}81)$$

M_{IC} = the bending moment at an interior column, in ft.-lbs.,
V = the total shear in the corresponding story, in lbs.,
h = the story height in ft., and
n = the number of aisles or panels.

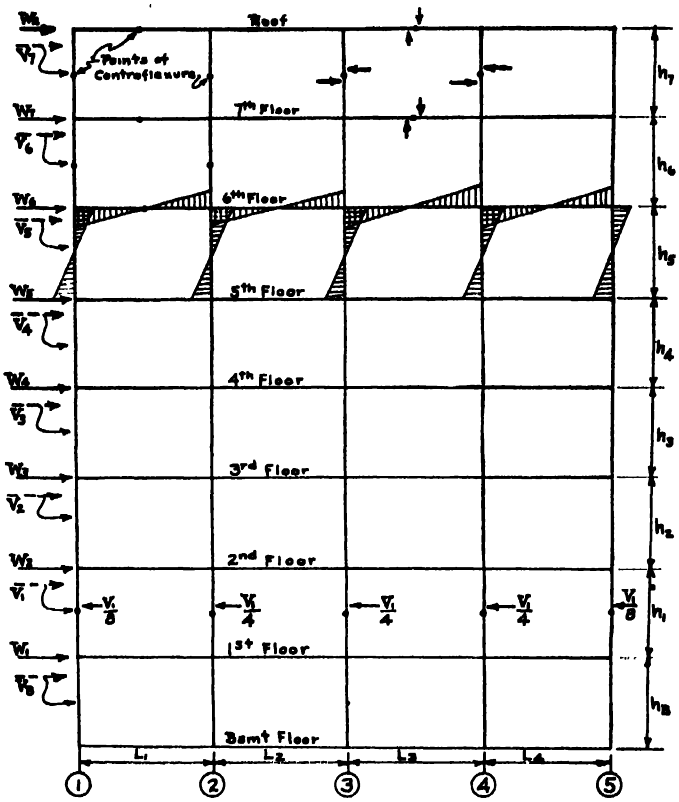


FIG. 507

(2) The bending moment at an exterior column in any story is equal to one-half that at the corresponding interior columns.

Expressed as a formula,

$$M_{OC} = \frac{W \cdot h}{4n}, \text{ in which} \qquad (S\text{–}82)$$

M_{OC} = the bending moment at an exterior column, in ft.-lbs.; and the other terms as defined for (S–81).

(3) The bending moment in a girder is a mean between the bending moments in the outside columns above and below the girder.

Expressed as a formula,

$$M_G = \frac{M_A + M_B}{2}, \text{ in which} \qquad (S\text{–}83)$$

M_G = the moment in a girder, in ft.-lbs.,
M_A = the moment in the outside column above, in ft.-lbs., and
M_B = the moment in the outside column below, in ft.-lbs.

(4) The direct compression in a girder is the algebraic sum of the wind panel load and the difference of the shears in the columns above and below the floor to the windward side of the girder under consideration.

Expressed as a formula,

$$C_G = W - \Sigma(V_B - V_A), \text{ in which} \qquad (S\text{–}84)$$

C_G = the compression in a girder, in lbs.,
W = the wind panel load at the floor, in lbs.,
V_B = the shear in a column below the floor, in lbs.
V_A = the shear in a column above the floor, in lbs.

(5) The direct stress due to wind in any exterior column (compression in leeward side and tension in windward side) **is calculated by taking moments of the wind panel loads above, about the point of contraflexure of the column.**

Expressed as a formula,

$$C = T = \Sigma\left[W_a \cdot \frac{h_a}{2} + W_b\left(\frac{h_a}{2} + h_b\right) + W_c\left(\frac{h_a}{2} + h_b + h_c\right) + \ldots\right] \div a, \qquad (S\text{–}85)$$

in which

$C = T$ = the direct tension or compression in an exterior column, in lbs.,
W_a, W_b = the wind panel loads in order, above the column,
h_a, h_b = the story heights in order above the column, and
a = the transverse width of the building.

Illustrative Prob. 319a. Calculate the bending moments in the typical members of the 5th floor in Fig. 507 for the following data:

W.L. = 25#/□′, spacing of columns in longitudinal direction = 20′-0″, spacing of columns in transverse direction = 16′-0″. 7th story height = 14′-0″ with 3′-0″ parapet. Typical story heights = 12′-0″, 1st story = 16′-0″, basement story = 11′-0″.

$W_R = (3 + 7) \times 20 \times 25 = 4500\#$ $V_7 = 4{,}500\#$
$W_7 = (7 + 6) \times 20 \times 25 = 6500\#$ $V_6 = 11{,}000\#$
$W_6 = (6 + 6) \times 20 \times 25 = 6000\#$ $V_5 = 17{,}000\#$
W_5 = do. $= 6000\#$ $V_4 = 23{,}000\#$
etc.   etc.

Shear in 5th story interior columns $= \frac{17{,}000}{4} = 4250\#$ each
" " " " exterior " $= \frac{4250}{2} = 2125\#$ "
" " 4th " interior " $= \frac{23{,}000}{4} = 5750\#$ "
" " " " exterior " $= \frac{5750}{2} = 2875\#$ "

Moment in columns ②, ③, and ④ above 5th floor,

$$M_{IC} = \frac{17{,}000 \times 12}{2 \times 4} = 25{,}500'\#$$

(This is the same value as 4250 × 6'-0" = 25,500'#)

Moment in columns ②, ③, and ④ below 5th floor,

$$M_{IC} = \frac{23{,}000 \times 12}{2 \times 4} = 34{,}500'\#$$

(This is the same value as 5750 × 6'-0" = 34,500'#)

Moment in columns ①, and ⑤ above 5th floor,

$$M_{OC} = \tfrac{1}{2}\,(25{,}500) = 12{,}750'\#$$

Moment in columns ① and ⑤ below 5th floor,

$$M_{OC} = \tfrac{1}{2}\,(34{,}500) = 17{,}250'\#$$

$$\text{Moment in 5th floor girders} = \frac{12{,}750 + 17{,}250}{2} = 15{,}000'\#$$

Direct compression in 5th floor girders.
Difference in shears 4th and 5th story exterior cols.

$$= 2875 - 2125 = 750\#$$

Difference in shears 4th and 5th story interior cols.

$$= 5750 - 4250 = 1500\#$$

Direct compression in girders:

①-② = 6000 − 750 = 5250#
②-③ = 6000 − 750 − 1500 = 3750#
③-④ = 6000 − 750 − 1500 × 2 = 2250#
④-⑤ = 6000 − 750 − 1500 × 3 = 750#

Tension in column ① in 5th story (= compression in column ⑤)

$$\frac{6000 \times 6 + 6500 \times 18 + 4500 \times 32}{64} = 4640\#$$

Tension in column ① in 4th story

$$\frac{6000 \times 6 + 6000 \times 18 + 6500 \times 30 + 4500 \times 44}{64} = 8390\#$$

Prob. 319b. Building 18 stories high, all stories 12'-0" high except 1st, which is 20'-0". Building 3 bays of 16'-0" each in transverse direction. Spacing of columns in longitudinal direction 20'-0". Parapet 3'-0" high. Wind load = 25#/□'. Make a line diagram of a typical cross-section of the building, showing the direct stresses in the columns and the values of the compression in the girders (express results in thousands of pounds). Make another line diagram showing the bending moments in all the columns and girders.

320. Design of the Members in General.

An important consideration in the design of the members of a building proportioned for wind pressure is that **the working stresses may usually be increased** in the average specifications. This is because wind stresses are only temporary, and often the actual wind pressure does not approach the unit pressure used in the design. For the exceptional cases, the factor of safety offers sufficient protection. Furthermore, the maximum combined stresses occur only at theoretical planes of extreme fibers, whereas the average stress on the cross-section is usually somewhat less.

SPECIFICATION CLAUSE*

When the stress in any member due to wind does not exceed 50 per cent of the stress due to live and dead loads, it may be neglected. When such stress exceeds 50 per cent of the stress due to live and dead loads, the working stresses prescribed in this chapter may be increased 50 per cent in designing such members to resist the combined stresses.

From the discussion in the preceding articles, it may be seen that the wind has a tendency to distort the frame as a whole. The important investigations are at the junction of the girders with the columns, as the bracing strength of the framing is governed by the strength of the connections, and usually not by the beam and column sections. Generally the column sections do not need to be increased over those required for the direct loads, as the direct wind stresses are usually less than 50% of the stresses due to direct load. The shears due to wind at the points of contraflexure of the columns are generally such that the areas of the cross-sections are more than ample to resist them safely, and this feature is commonly not investigated. Also, the girder sections do not usually have to be increased over those required for the regular load, as the bending stresses due to wind are usually less than 50% of those caused by the direct load. The direct compressive stresses generated in the girders by the wind are generally small compared with the bending stresses, and this investigation is also usually omitted. The floor construction tends to relieve the girders of this stress, so that additional safety is secured. Summarizing the above discussion, **the connections of the girders to the columns in a building designed for wind action must usually be increased over those necessary for ordinary cases, and this is the most important feature of the detail design.**

321. Design of Columns.

As stated in Art. 320, the column cross-sections usually do not need to be increased over those required for the direct loads, as the stresses due to wind are generally less than 50% of those caused by the direct loads. If a case does occur in which the wind stress exceeds one-half of the direct stress, it is necessary to make an investigation.

The bending moment due to wind on any column may be investigated in a similar manner to the moment due to eccentric loads (Art. 243). The additional compression may be computed from $s = M \cdot c \div I$. Some engineers prefer to calculate the equivalent central load (the direct load which will produce the same unit stress). The critical section will occur as illustrated in Fig. 508. The horizontal shear (assumed to be carried by the column at its point of contraflexure) multiplied by its arm (its distance to the section) gives the resulting bending moment. The equivalent central load due to wind moment, P_w, may be expressed by

$$P_w = \frac{H \cdot e \cdot c}{r^2}, \text{ in which} \qquad (S\text{–}86)$$

H = the horizontal shear at the point of contraflexure, in lbs.,

* The Code of Ordinances of the City of New York.

e = its arm to the critical section in ins.,
c = the distance to extreme fibre in compression, in ins., and
r = the radius of gyration of the column cross-section in ins., about the axis perpendicular to the direction of bending.

In addition to the bending stress in the exterior columns, an axial stress occurs. The governing cases are those on the leeward side, as additional compression is developed, whereas tension is generated on the windward side. This must be added to the other stresses. Thus for an outside column, the maximum stress is the sum of the stress due to direct load, that due to bending and the resultant axial wind stress. An interior column receives no axial wind stress, but the bending is twice that on an outside column so that each typical case must be investigated separately.

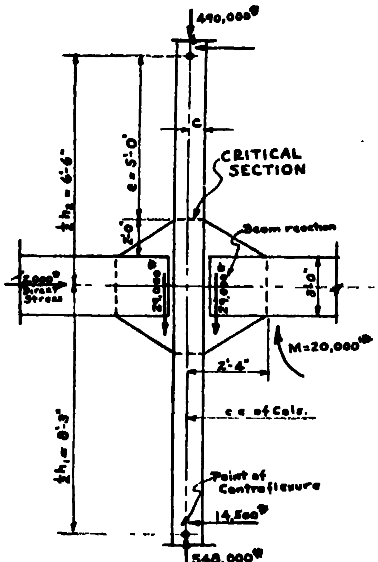


FIG. 508

The horizontal shear on any section (acting at the point of contraflexure) is comparatively small and the shearing intensity is usually amply safe and is generally not investigated. The uplift, if any (on the lowest tier of the windward columns), should be investigated. This is the result of subtracting the dead load from the direct stress. Any uplift should be provided for by anchor bolts. Column splices should also be checked up for stresses induced in them. The best type of splice to resist wind action is with the milled ends and side plates (Art. 259). Splices with cap plates and clip angles offer much less resistance to bending. The columns should be made structurally continuous from the foundations to the roof for best results.*

322. Design of Girders.

As stated in Art. 320, the girders usually do not need to be increased over those required for the ordinary loads, as the bending stresses due to wind are generally less than 50% of those caused by the regular loads. If a case does occur in which the wind stress exceeds one-half of that due to the direct load, it is necessary to make an investigation.

* Staggered columns cause complicated wind calculations.

In the usual case of loading, light end connections do not fix the ends of the girder to any great extent, and the bending moment is based upon a simple span, as illustrated in Fig. 509 (*a*). However, when relatively heavy end connections occur, as is often the case in wind bracing, there is more end restraint, although the beam does not have fully fixed ends. When bracket types of connections are used, especially with

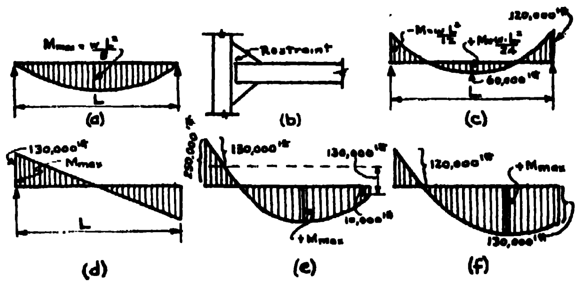


FIG. 509

gusset plates, as in Fig. 509 (*b*), the effective depth at the ends of a girder is increased. This would have a tendency to increase the negative moment and to decrease the positive moment, so that a fixed end girder is not entirely an unreasonable assumption, as shown in Fig. 509 (*c*). The wind moment is assumed to be of straight line variation, as illustrated in (*d*), with the maximum value occurring at the ends. The two cases, (*c*) and (*d*), may be combined to attain the maximum values of the negative and positive moments, as shown in (*e*). However, a practical consideration enters. The two end connections are made alike of course, to provide for a reversal of wind direction. The columns will generally be about the same size at each end so that equal stiffness may be assumed for each end of the girder. On this basis, the diagram in Fig. 509 (*e*) may be shifted so that the positive moment at the right is of the same value as the negative moment at the left. This is illustrated in (*f*). The girder would then be designed for the maximum value determined from diagrams (*f*) or (*c*), with the maximum positive moment probably not at mid-span.

The above procedure is too complicated to use ordinarily in practice, especially if concentrated loads are involved, and practically the same maximum values may be obtained by considering the girder as simply supported (in spite of any heavy end connections) and superimposing the wind moment on the vertical load moment diagram, as illustrated in Fig. 510 (*a*) and (*b*).

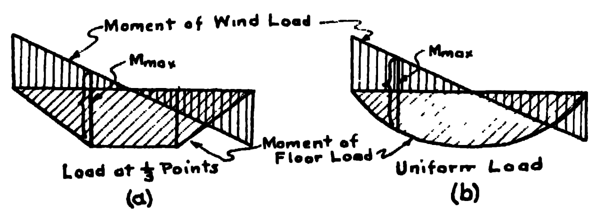
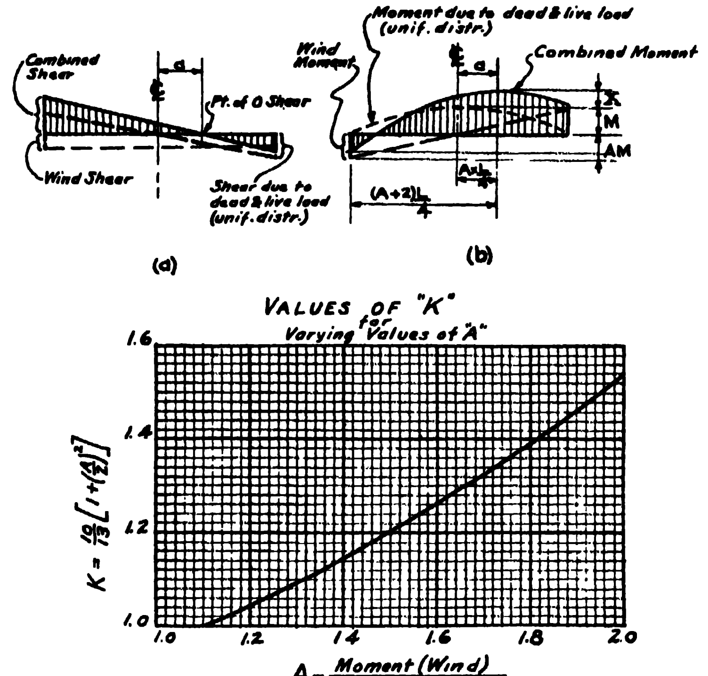


FIG. 510

Another method of analyzing the maximum moment in girders with reference to the Philadelphia Building Ordinance follows. Figure 511 (*a*) shows the combined shear (solid lines) due to dead and live load and wind. The dotted lines show the separate influences, based upon a uniform vertical load and the usual assumption of simply supported beams. Figure 511 (*b*) shows the combined moment (plotted opposite to that in Fig. 510) in a similar manner. The figures represent the algebraic

sum of the shears and moments respectively. The combined moment curve is a parabola with its maximum ordinate at some point other than mid-span. The maximum value occurs at the point of zero shear. The value will exceed the live and dead load moment by some amount, such as X, shown in Fig. 511 (b). The wind moment may be represented by some function of the dead and live load moment, as $AM = A\,(w \cdot L^2 \div 8)$. The wind shear may be expressed as

$$\frac{\text{Wind Moment}}{\frac{L}{2}} = \frac{A(w \cdot L^2 \div 8)}{\frac{L}{2}} = \frac{w \cdot L}{4} \cdot A.$$



WIND GIRDER DESIGN (Phila. Bldg. Ordinance)
A less than 1.1, design for L.L & D.L Moment
A between 1.1 and 2.0, design for K(L.L & D.L. Moment)
A greater than 2.0, design for 10/13 Wind Moment
All at regular fibre stresses

(c)

FIG. 511

The distance to the point of 0 shear from mid-span is

$$\frac{w \cdot L}{4} \cdot A \div w = A \cdot \frac{L}{4}.$$

The distance to the point of 0 shear from the left-hand end of the beam is then

$$A \cdot \frac{L}{4} + \frac{L}{2} = (A + 2)\frac{L}{4}.$$

By the properties of the parabola,

$$\frac{X}{X + M + AM} = \frac{A^2\left(\frac{L}{4}\right)^2}{(A + 2)^2\left(\frac{L}{4}\right)^2} = \frac{A^2}{(A + 2)^2}.$$

From this expression,

$$X = M\frac{A^2 + A^3}{4A + 4}.$$

The maximum moment is then $M + X$, or

$$M + X = M + M\frac{A^2 + A^3}{4A + 4}, \text{ or}$$

$$M + X = M\left(1 + \frac{A^2 + A^3}{4A + 4}\right).$$

The Philadelphia Building Ordinance allows a 30% increase in the allowable stress for combined wind stresses. Therefore, unless X is more than 30% of M, the design of the girder is governed by the live and dead load moment. When $X = 0.3\,M$, $A = 1.1$. In other words, **unless the wind moment is 110 per cent of live and dead load moment, the design is governed by the latter.**

For values of A exceeding 1.1, the design is governed by $\frac{10}{13}$ $(X + M)$ at the usual fibre stresses.* The moment to design for may be expressed as

$$\frac{10\,M}{13}\left(1 + \frac{A^2 + A^3}{4 + 4A}\right).$$

This may be expressed as KM. Figure 511 (c) gives a diagram for determining values of K for varying values of A.

Illustrative Prob. 332a. If wind moment = 116,000′# and moment due to dead and live loads is 84,000′#, what moment should the girder in question be designed for at usual fibre stresses in accordance with the Philadelphia Building Ordinance?

$\frac{116{,}000}{84{,}000} = 1.38$. From Fig. 511 ($c$), when $A = 1.38$, $K = 1.117$. The moment to be designed for at 16,000# □″ is then $1.117 \times 84{,}000 = 93{,}800$′#.

323. Design of Connections.

The important detail design in providing for the resistance to wind pressure is that of the connections of the girders to the columns, as the bracing strength of the whole structure is governed by the strength of the connections, and not by the beam sections.

The end connections must be designed to resist the wind load moments at the ends of the girders, with the connections at each end made the same, to provide for a reversal of wind direction. The ordinary beam connections provide some resistance to such moments, and for small values of the wind moments, particularly in the upper stories, they may be altered slightly and still give sufficient resistance.† **All the connections must be made by rivets, to be efficient.** The top clips are usually made the same size as the bottom seat angles, so that a balanced couple will result. The seat angle type of connection is adopted in such cases, in order to approach a bracket design, as the ordinary connection angles

* The ratio of the allowable stresses, common cases and combined wind is 10 to 13.

† Some engineers believe that the full value of the connections of girders to columns should not be assumed, and hence they design the connections quite liberally. They claim that a great number of connections would not all work at the same time, particularly for irregularly shaped buildings. The authors believe that the usual values of the rivets may be used in such cases, as the wind loads are only temporary.

attached to the webs of the girders (Art. 28) are not very efficient in resisting moment, because they are near the middle of the beam depth. Hence connections to the flanges of the girders are used, as greater resisting arms result.

Any tendency to distort the frame as a whole causes shear in the rivets connecting the girder flanges. The controlling value of the rivets is governed by bearing or single shear. If usual thicknesses of metal are employed, **single shear will control the strength of the rivets. The value of the rivets connecting a flange to the clip angle times the depth of the beam equals the resisting value of the connection against wind moment.** This is equivalent to the moment of a resisting couple. For very small moments, two rivets in each flange, as in Fig. 512 (*a*), may be sufficient. This requires $3\frac{1}{2} \times 3\frac{1}{2}$

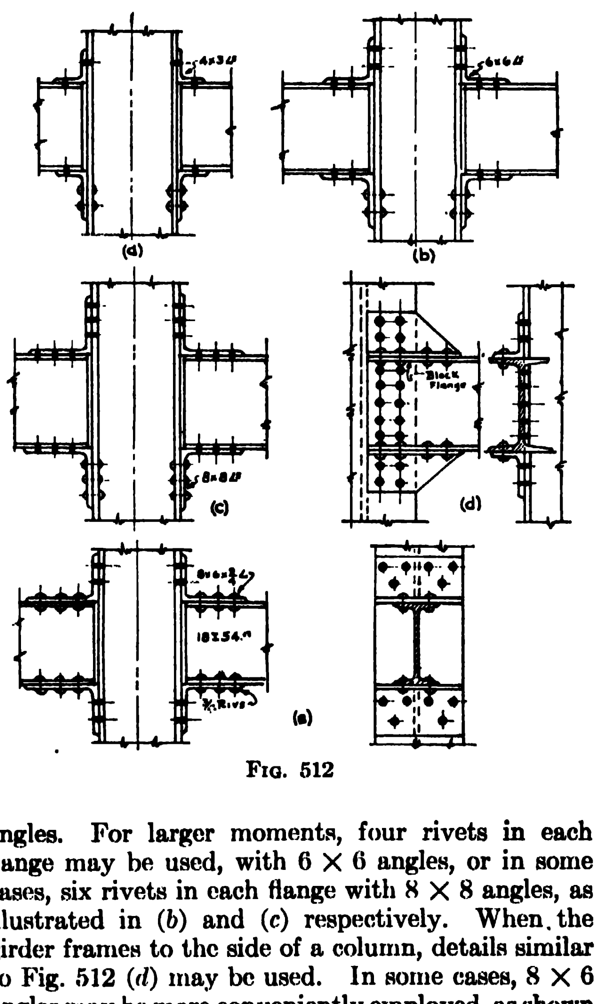


FIG. 512

angles. For larger moments, four rivets in each flange may be used, with 6×6 angles, or in some cases, six rivets in each flange with 8×8 angles, as illustrated in (*b*) and (*c*) respectively. When the girder frames to the side of a column, details similar to Fig. 512 (*d*) may be used. In some cases, 8×6 angles may be more conveniently employed, as shown in (*e*). The rivets in the vertical legs of the connection angles should be at least the same in number, as the moment must be transferred from the girder to the clip angles, and from the latter to the column. When this number is maintained, the connection will not be controlled by the rivets in the vertical legs of the clip angles, as the lever arm of the resisting couple, based on the latter, is greater.

The rivets in the seat must of course be strong enough to carry the beam reaction, as in any ordinary case, and stiffeners may be necessary under the seat angle (Art. 254). The strength of the connection to offset the wind moment is also limited by the bending resistance of the clip angles. This feature should be investigated and angles of sufficient thickness should be employed. Side clips are sometimes added to the connections, to furnish extra stiffness, although they do not supply much wind resistance, as the girders are usually not deep enough to have many of their connecting rivets at appreciable lever arms.

Illustrative Prob. 323a. If the bending moment due to wind in a girder is 20,000′# and the size is 20 I 65.4, what type of connection is necessary?

Let n = the number of ¾″ rivets connecting one flange. Single shear value of 1-¾″ field rivet = 4420#

$$20{,}000 \times 12 = n \times 4420 \times 20$$
$$n = 2.7+$$

Use 6×6 angles and 4 rivets in each leg (see Fig. 512(*b*)).

When larger wind moments occur in the girders, clip angles for the connections may not be sufficient. As 8×8 angles are the largest size available, the resistance of 6 rivets at their lever arm is the maximum. Clip angles can develop only a small part of the capacity of a beam, whereas brackets can be made to develop the entire net bending resistance of a beam, if necessary. In the lower stories, where considerable wind moment may be generated, bracketed connections become more general. Figure 513 (*a*) and (*b*) shows two common types, that in (*a*) when I beams are involved, and (*b*) when plate girder sections are used. These are generally made up of gusset plates and angles. The slopes are commonly made at 45°, to clear openings, intermediate walls, and so on. In designing, it is customary to neglect the strength of the gusset as a whole and to count upon the angles and that portion of the gusset which is confined between the angles. Stresses of diagonal tension and diagonal compression are set up as indicated by the dotted lines in Fig. 513 (*b*). The net section of the angles and gusset between them should be sufficient to resist the tension. Stiffener angles are usually necessary on the compression side, and should always be used when the length of the diagonal edge exceeds 30 times the thickness of the gusset. The gusset is commonly made ⅜″ thick, which is consistent with the web thicknesses of average I beams and plate girders. The width of the angles "A" in Fig. 513 (*b*) adjacent to the gusset plate is made enough to accommodate

the rivets, — usually 3″. The width of the outstanding legs may be whatever is required, as long as proper fabrication and erection clearances are maintained.

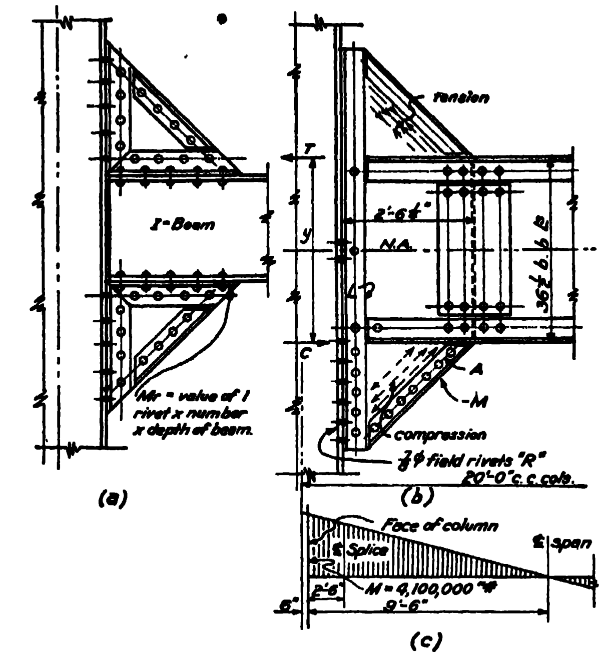


FIG. 513

Illustrative Prob. 323b. Design the joint in Fig. 513 (*b*) if $M_{max} = 4{,}100{,}000''\#$.

Use connection angles "*L*" to face of column $4 \times 4 \times \frac{1}{2}$. Average stress on rivets "*R*" is one-half the maximum allowable.

Single shear ⅞″ field rivet = 6010#.

Assume 30 rivets in each leg spaced 3″ o.c.

Center of gravity of each group above and below is at T and C respectively = 23½″ from N.A.

$$C = \frac{6010 \times 30}{2} = 90{,}200\#$$

Arm between C and T = $2 \times 23\frac{1}{2} = 47''$

$M_r = 90{,}200 \times 47 = 4{,}240{,}000''\#$ O.K.

Shop rivets in angles "*R*" in double shear

Double shear ⅞″ shop rivet = 14,430#

$$C = \frac{14{,}430 \times 14}{2} = 101{,}000\#. \quad \text{Arm} = 2 \times 21''$$

$$M_r = 101{,}000 \times 42'' = 4{,}250{,}000''\# \text{ O.K.}$$

The thickness of the gusset must be sufficient to develop the full strength of the rivets. Bearing of rivets on the two angles O.K. Enclosed bearing on gusset

$$14{,}430 = 30{,}000 \times \tfrac{7}{8} \times t$$

$$t = 0.55'' \qquad \text{Use } \tfrac{5}{8}'' \text{ gusset.}$$

The moment at the splice between the gusset plate and the girder web may be determined by proportion, since the wind moment is assumed as straight line variation.* Thus in Fig. 513 (*c*),

$$M \text{ at splice} = 4{,}100{,}000 \times \frac{7.0}{9.5} = 3{,}030{,}000''\#$$

The splice plates may be proportioned for this moment and the shear at this point by the method outlined in Art. 73.

$$\text{Flange stress in girder at splice} = \frac{3{,}030{,}000}{36.5} = 82{,}900\#$$

Tension in angles at edge of gusset at top

$$\frac{82{,}900}{0.707} = 117{,}000\#$$

$$\text{Net area required} = \frac{117{,}000}{16{,}000} = 7.33\square''$$

$$\begin{aligned} \text{Gross area } 2\ \llcorner\ 3\tfrac{1}{2} \times 3\tfrac{1}{2} \times \tfrac{5}{8} &= 7.96 \\ 2 \text{ holes } \tfrac{5}{8} \times 1 &= 1.25 \\ \hline & 6.71\square'' \\ \text{Gross area } 3\tfrac{1}{2} \times \tfrac{5}{8} &= 2.19 \\ 1 \text{ hole } \tfrac{5}{8} \times 1 &= 0.62 \\ \hline & 1.57 \end{aligned}$$

Net area = 6.71 + 1.57 = 8.28□″ O.K.

Use $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{8}$ edge ∟.

$$\frac{117{,}000}{14{,}430} = 8.1 \qquad \text{Use 9 rivets.}$$

Compression = 117,000#. Length of diagonal = 43″

r of 2 ∟ $3\frac{1}{2} \times 3\frac{1}{2}$, ⅝″ back to back = 1.05″ (average)

$$p = 16{,}000 - \frac{70 \times 43}{1.05} = 13{,}140\#/\square''$$

$$\text{Gross area required} = \frac{117{,}000}{13{,}140} = 8.91\square''$$

$$\begin{aligned} \text{Gross area } 2\ \llcorner\ 3\tfrac{1}{2} \times 3\tfrac{1}{2} \times \tfrac{5}{8} &= 7.96 \\ \text{Gross area } 3\tfrac{1}{2} \times \tfrac{5}{8} &= 2.19 \\ \hline & 10.15\square'' \text{ O.K.} \end{aligned}$$

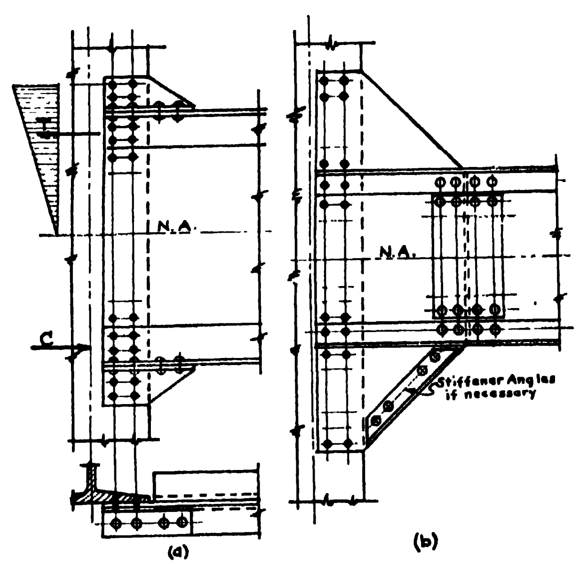


FIG. 514

* The moment at the connection may be considered as slightly less than the theoretical value at the center-line of the column, since it is one-half the column-width away from the center-line.

The bending in the connection angles "L" should be investigated. The value of a field rivet times its distance to the fillet of the angle is the local bending. The resistance is based upon the thickness of the angle and a length corresponding to the spacing of the field rivets. For usual cases, if the gauge of the rivets does not exceed $2\frac{1}{2}''$, a $\frac{5}{8}''$ angle is sufficient. If a gauge of $4''$ occurs, a $1''$ angle is required.

In some cases, the web of the girder may connect to the flange of the column. A connection similar to that in Fig. 514 (*a*) may be used, or if a large wind moment occurs, a gusset may have to be introduced, as in (*b*). Such connections are satisfactory if the girders have considerable depth. The values of the rivets vary as their distances from the neutral axis, and the resisting moment is calculated as previously discussed, — by finding the center of gravity of each group, and with the average stress on a rivet times the number in the group on one side of the neutral axis, times the lever arm of the two forces is the resisting couple (Art. 73). When the building is quite wide (say, more than 5 bays), some engineers use bracket connections at the outside and first interior columns, and then use heavy connection angles at the other interior columns, claiming that such interior connections offer sufficient resistance to the local wind stresses, as the effect of the wind is considerably dissipated by the first heavy connections and the floor system.

Prob. 323c. Design a connection of the type in Fig. 512 (*a*) for the data of Illustrative Prob. 323*b*. Make a sketch of the joint at a scale of $\frac{3}{4}'' = 1'\text{-}0''$ if the column is a 14 BH.

Prob. 323d. Design a connection similar to Fig. 513 (*a*) if the I beam is a 24 I 100 and the column a 12 BH. Wind moment = 1,200,000″#.

Prob. 323e. Design a connection similar to Fig. 513 (*b*) if the plate girder is $30\frac{1}{2}''$ back to back of angles with $5 \times 3\frac{1}{2} \times \frac{1}{2}$ flange angles. $M = 2{,}300{,}000''\#$. Column 12 BH. Use $\frac{3}{4}''$ rivets.

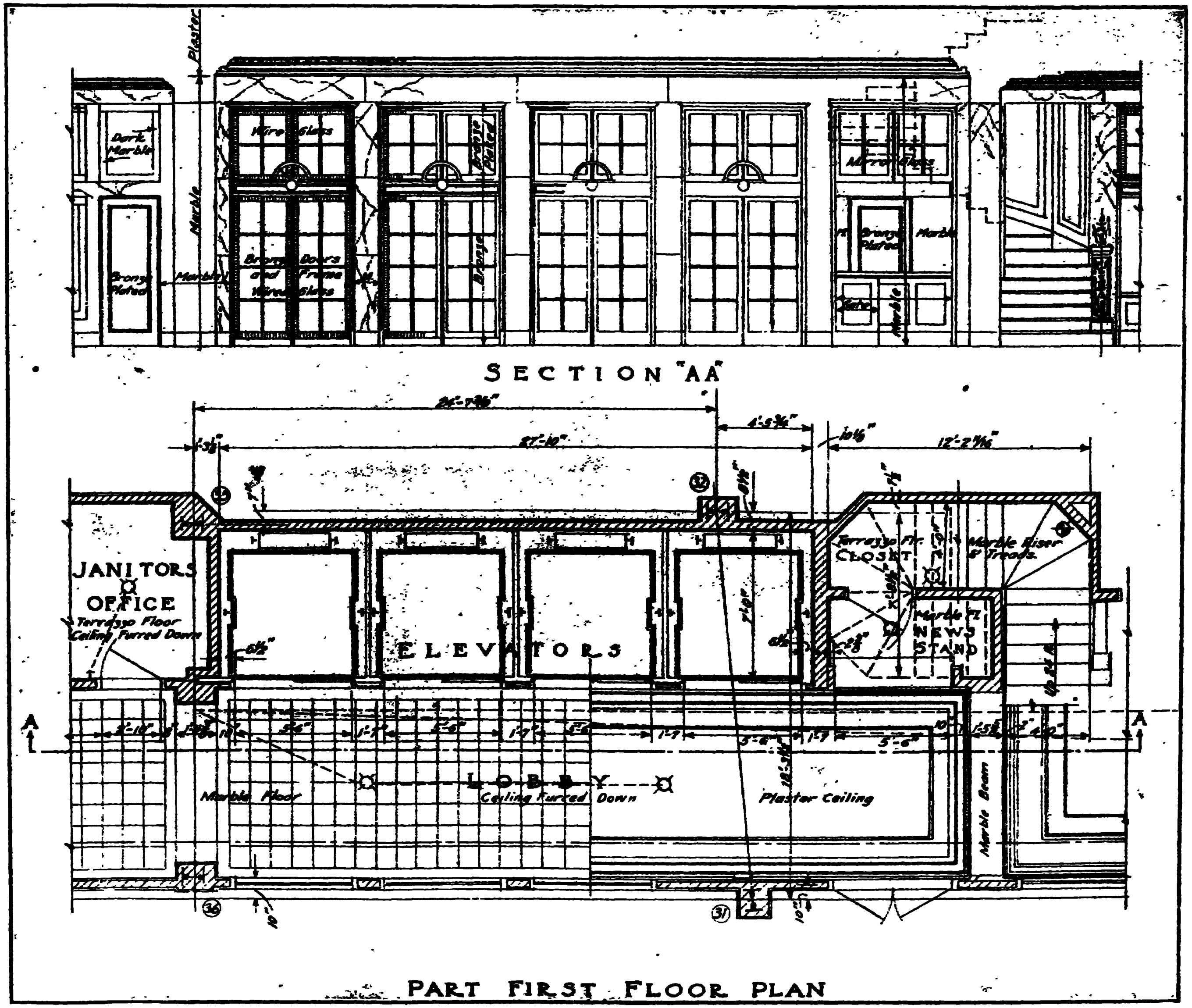


PLATE 38 ARCHITECTS' PLAN AND ELEVATION OF ELEVATOR LAYOUT
RICE BUILDING — PARKER, THOMAS AND RICE, ARCHITECTS

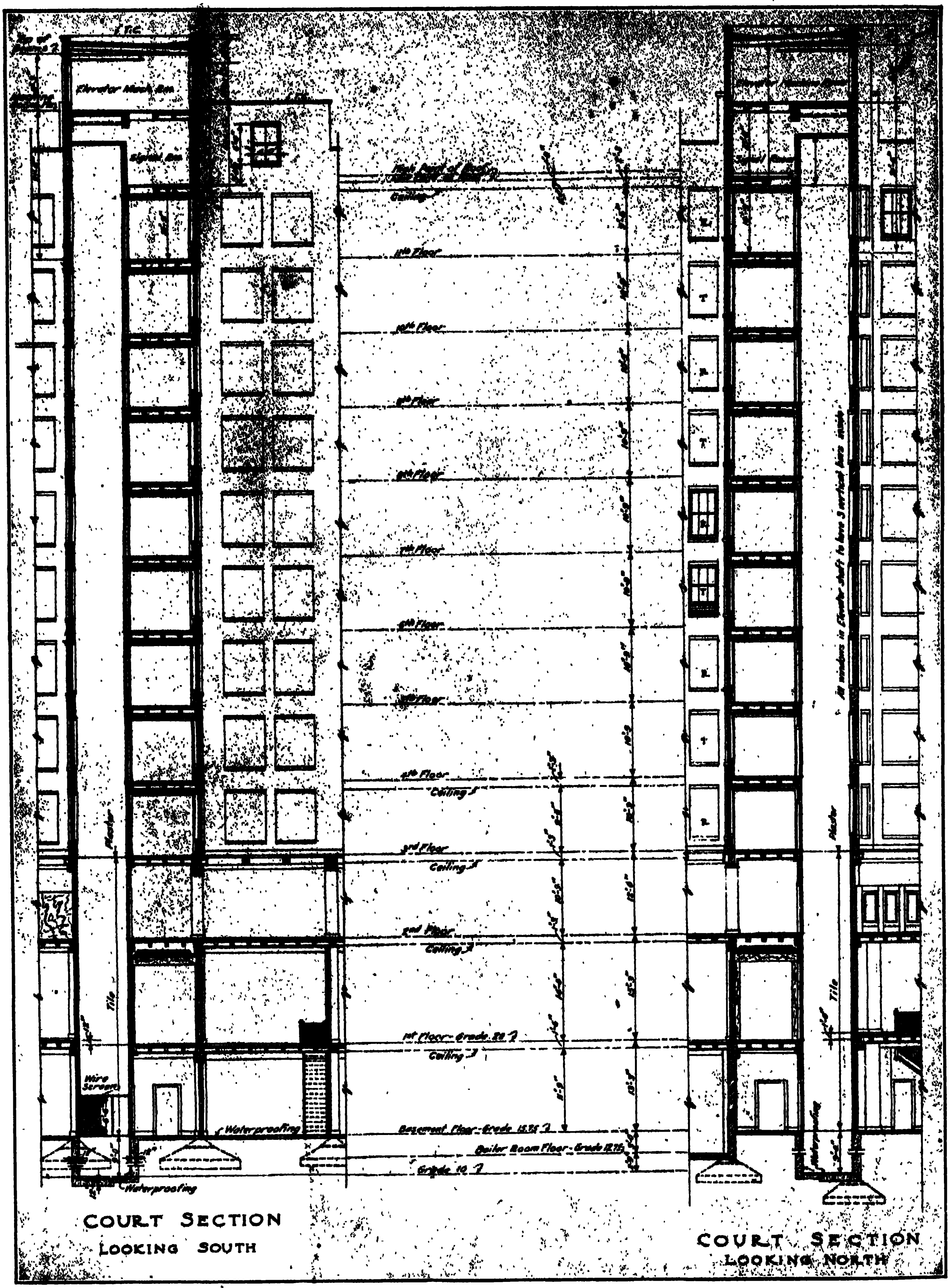


PLATE 39 ARCHITECTS' SECTION OF ELEVATOR SHAFTWAY
RICE BUILDING — PARKER, THOMAS AND RICE, ARCHITECTS

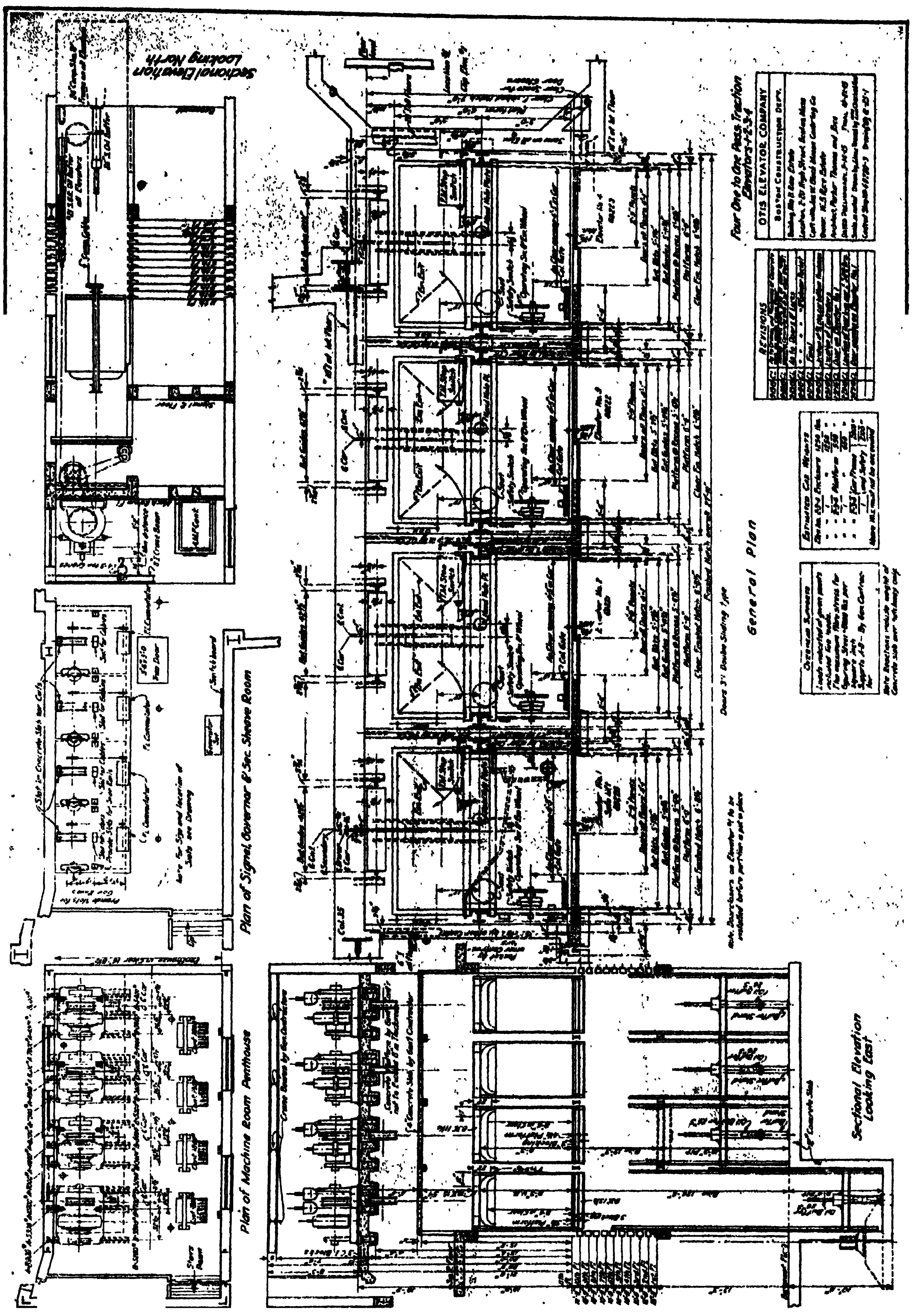


PLATE 40 ELEVATOR LAYOUT — RICE BUILDI
PARKER, THOMAS AND RICE, ARCHITECTS

CHAPTER 28

EVATOR FRAMING*

324. Structural Design Involved.

The planning of elevator service naturally starts with the architect. He must decide, in conference with his client, how many cars are to be used, the type of service (freight or passenger),† where the hatchways are to be located, what floors are to be served, the size of car desired, and, by using standard clearances (based upon other installations), the approximate dimensions of the hatchways. With such data, tentative, typical plans and sections are drawn, similar to those illustrated by Pls. 38 and 39.

The next step of the architect usually is to call in representatives of one or more concerns that manufacture and install elevators. The detail and final arrangement of the sizes of cars, clearances (and therefore dimensions of hatchways), locations of sheave beams, counterweight, guides, design of elevator machine, motor, gearing and roping is largely the province of the elevator company (Fig. 515). It is a specialized field of work and even mechanical engineers who are planning the bulk of other equipment, leave such work to the elevator companies. Hence it is not unreasonable for an architect to desire expert advice in this connection from the elevator engineers.

The representative of the elevator manufacturer makes a study of the particular layout in order to arrive at the most efficient service, even suggesting an increase or rearrangement of the service. One means of determining the surrounding conditions for large jobs, in addition to the architect's tentative plans and sections, is the use of a questionnaire. The following represents a typical one:

(1) What is the location of the building — giving city, streets and general character of environment?
(2) What are the horizontal transportation facilities affecting the flow of people in and out of the building?
(3) How are the horizontal transportation facilities located with reference to the building entrances? State distance away and relative location of various stations, car lines, parking space, etc.
(4) What is the plot plan of the building — showing the size and general arrangement of the ground floor and the location of the elevators, if that has been tentatively determined?
(5) What are the typical floor plans of the building?
(6) How many floors has the building?
(7) What are the heights of the various floors?
(8) What is the net rentable area by floors?
(9) What kinds of businesses are to be housed?
(10) What sizes of businesses?
(11) What is the location of the businesses on floors?
(12) What is the expected population by floors?
(13) What is the percentage of clerical help?
(14) What is the time at which the various businesses start work?
(15) What is the time of stopping work?
(16) What are the conditions affecting traffic flow in the noon period?
(17) Do the elevators serve the basement?
(18) What is the expected percentage of absences?
(19) What is the expected percentage of vacancies?
(20) What interval is desired and what limits of interval would be considered satisfactory?
(21) What is the maximum number of passengers to be handled in a five-minute peak in the morning?
(22) What is the maximum five-minute peak of passengers at noon?
(23) What is the maximum five-minute peak of passengers at night?
(24) What is the flow of outside population entering and leaving the building, particularly in relation to the peak period?
(25) What is the expected inter-floor traffic, noting any special inter-floor requirements?
(26) How are the toilet facilities located?
(27) How are the restaurants located?
(28) What is the flow of restaurant service and the maximum five-minute peak period going to and coming from restaurants?
(29) What is the location of assembly rooms in the building, if any?
(30) What is the general discipline and control of the population, stating any special requirements of discipline?
(31) What special requirements affecting elevators are there in the building?
(32) What structural or other interference is there affecting location of elevators?

The extent to which such questions are gone into depends upon the size of the job. Of course such minute detail is hardly necessary where only one or two cars are to be installed.

With the data obtained from the architect, the elevator company studies the installation from a standpoint of flow of traffic, particularly as to peak loads, influx, noon traffic and outflow, inter-floor traffic, density of population, size of car, hatchway doors and car gates, interlocks, number of cars in

* Some of the material and figures in this chapter are derived from notes and drawings furnished by the Otis Elevator Company.
† For a discussion of escalators, see Art. 312.

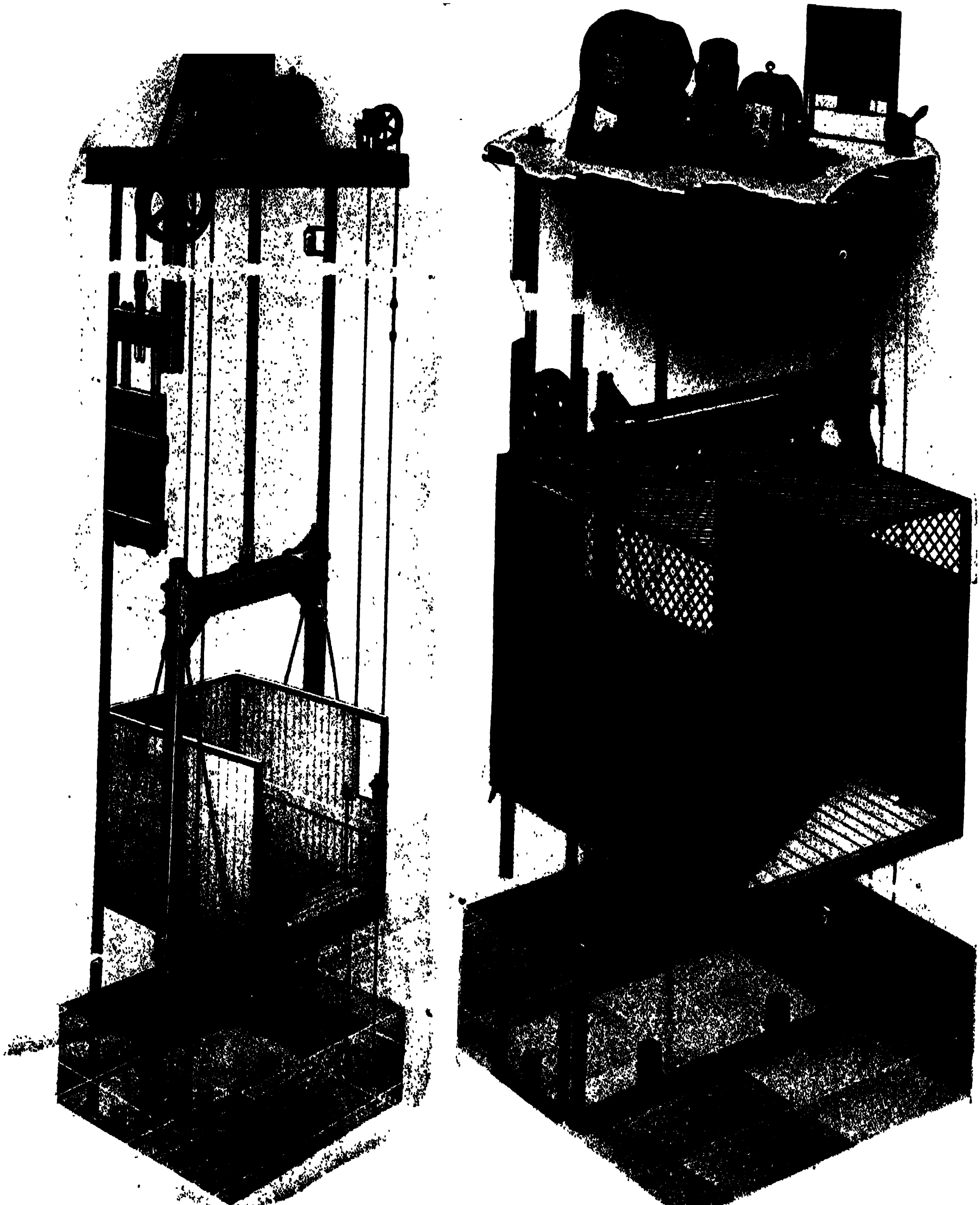


FIG. 515. GEARED FREIGHT ELEVATOR—SINGLE WRAP TRACTION*

(a) 1 to 1 Roping—Steel Guides

(b) 2 to 1 Roping—Steel Guides

* Courtesy Otis Elevator Co.

a bank and number of banks, time interval, express and local banks, elevator speeds, determination of round trip time, signals, and so on. The type of elevator is selected as to the traction, roping, control, dimensions and travel, load and speed, and so on. The layout then more or less settles the arrangement of guide posts and guides, sheaves and sheave beams, counterweight, driving machine motor and controller. This is consummated in an **elevator drawing**, similar to that shown on Pl. 40.* Figure 516 also shows the essentials of another layout.

be familiar with the essentials of the requirements and to have a general knowledge of the problem.

325. Types of Elevators.

Elevators used in modern buildings are generally of the electric type and are sub-divided into four kinds, namely

(*a*) drum
(*b*) worm-gear traction
(*c*) helical gear
(*d*) gearless traction
  (1) direct drive (one-to-one) (Fig. 517)
  (2) two-to-one.

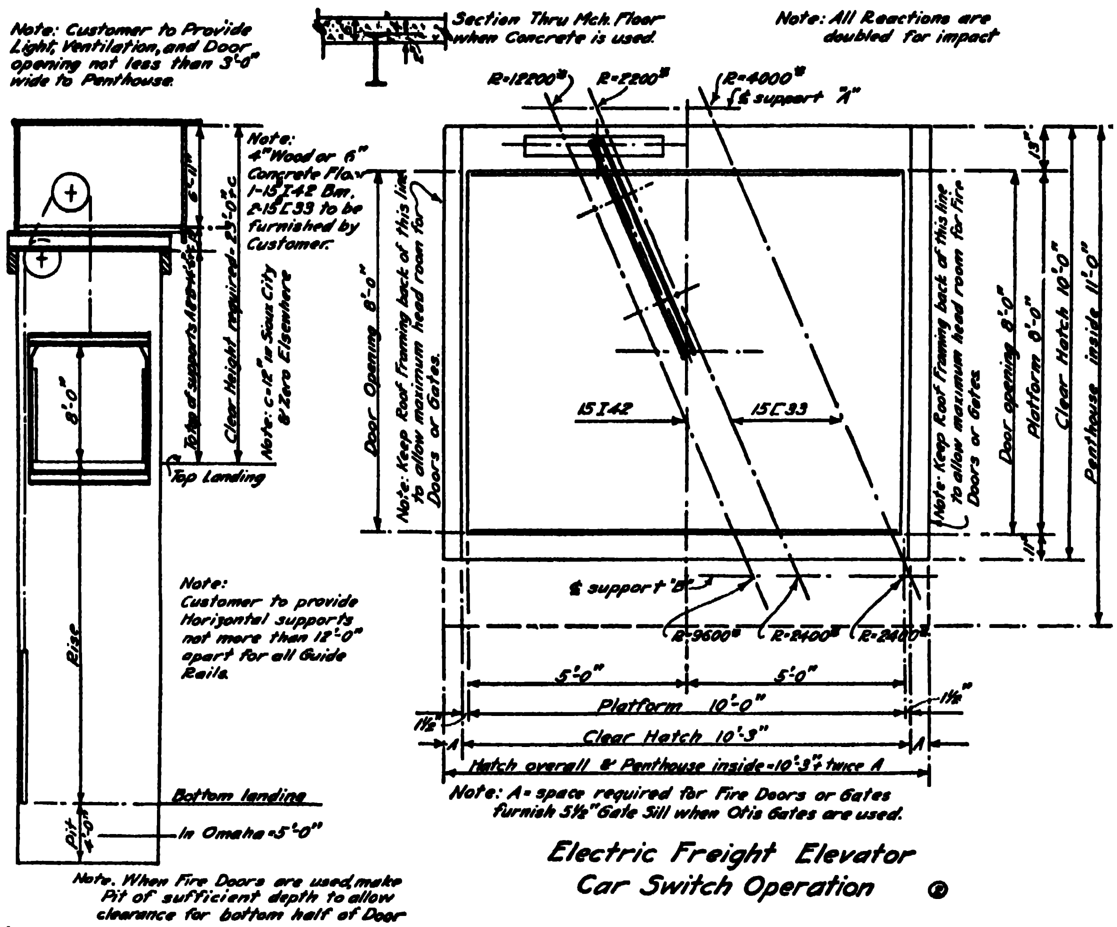


Fig. 516. Freight Elevator*

1 to 1 Roping — Car Switch Operation

It is from such a drawing, furnished by the elevator company, in addition to the architectural plans and sections, that the structural engineer obtains the information necessary to provide proper supports for the elevator equipment. Although the structural engineer is not required to design the sheave beams and other intermediate supports, as described above, it is a distinct advantage to him to

Some office buildings, and buildings of a similar character, are at present equipped with hydraulic elevators, although this type of equipment is now being rapidly replaced by the electric machine. The hydraulic elevators may be either:

(*a*) vertical geared hydraulic
(*b*) horizontal geared hydraulic
(*c*) direct lift plunger
(*d*) inverted high pressure plunger.

The fact that about nine-tenths of the elevators in service are electric seems to bear out the contention that speed can

* Courtesy of Otis Elevator Co.

be obtained with commensurate safety and that electric elevators are usually preferred because of their flexibility in service.

Of all of these, the electric gearless traction elevator with the direct one-to-one drive seems to be the most popular for general office building service. For purposes of simple comparison, the usual types of both electric and hydraulic elevators are shown in Fig. 518, in which (*a*) illustrates the double wrap, gearless traction, one-to-one roping; (*b*) the double wrap, gearless traction, two-to-one roping; (*c*) the geared electric overhead drive; (*d*) the geared electric basement drive; (*e*) the vertical plunger hydraulic; and (*f*) the direct-acting plunger. The difference between the electric geared and gearless machines lies in the fact that high-speed motors are more economical in first cost and operation than low-speed machines. In order to provide a slow moving car, gears are used to reduce the motor speed to a proper cable or car speed. Even the electric gearless traction equipment is much more economical of space, initial cost and operation than the hydraulic. **Space is an important feature for consideration in buildings** and the electric machine not only eliminates the necessity of pressure and discharge tanks, together with the pumping equipment, but is also a great improvement due to the increased speeds obtainable at a constant acceleration. The increased height of many modern buildings has also made the hydraulic plunger elevator an unsafe engineering problem for such cases.

For the purpose of this discussion the electric elevator only will be considered and much of the information is based upon the advice of the Otis Elevator Company of New York. It is, however, interesting to designate the difference between the usual trade names. In general, electric elevators are classed as gearless or geared. In the gearless type a slow-speed motor drives the sheave directly from the motor shaft with a double-wrap hook-up. In the geared machine a high-speed motor, with its speed reduced through a worm and gear, drives through a sheave fastened on the gear shaft. Where an elevator is so roped that it represents a direct suspension in which the rate of motion of the car is equal to that of the supporting cable and the cable is attached directly to the car frame, it is called a one-to-one roping. If, however, the rate of motion of the car and counterweight is one-half that of the cable, and a secondary sheave is used on top of the bars and the counterweights, it is called a two-to-one roping. When a secondary sheave is used to effect a second wrapping of the driving sheave the mechanism is known as double-wrap. **The ordinary elevator, however, is fitted only with a single-wrap machine.** Figure 519 illustrates a typical layout of this kind.

326. Horizontal Clearances.*

With the size of car fixed in plan (see p. 482), the clear dimensions of the hatchway are determined by adding clearances between the car platform and the floor threshold and counterweights.

SPECIFICATION CLAUSES †

Thresholds, Projections and Recesses (*a*) Strong and substantial beveled metal or wood plates shall be located under all thresholds, beams and other fixed construction which projects into shaftways one inch (1″) or more beyond the general line of the shaftways of the elevator on sides where there are car openings. Metal plates, when not backed with wood, shall be made of not less than No. 12 B & S gauge metal. Recesses in shaftway enclosures on sides where

* Vertical clearances are considered in Art. 327.
† Regulations of the State of Massachusetts.

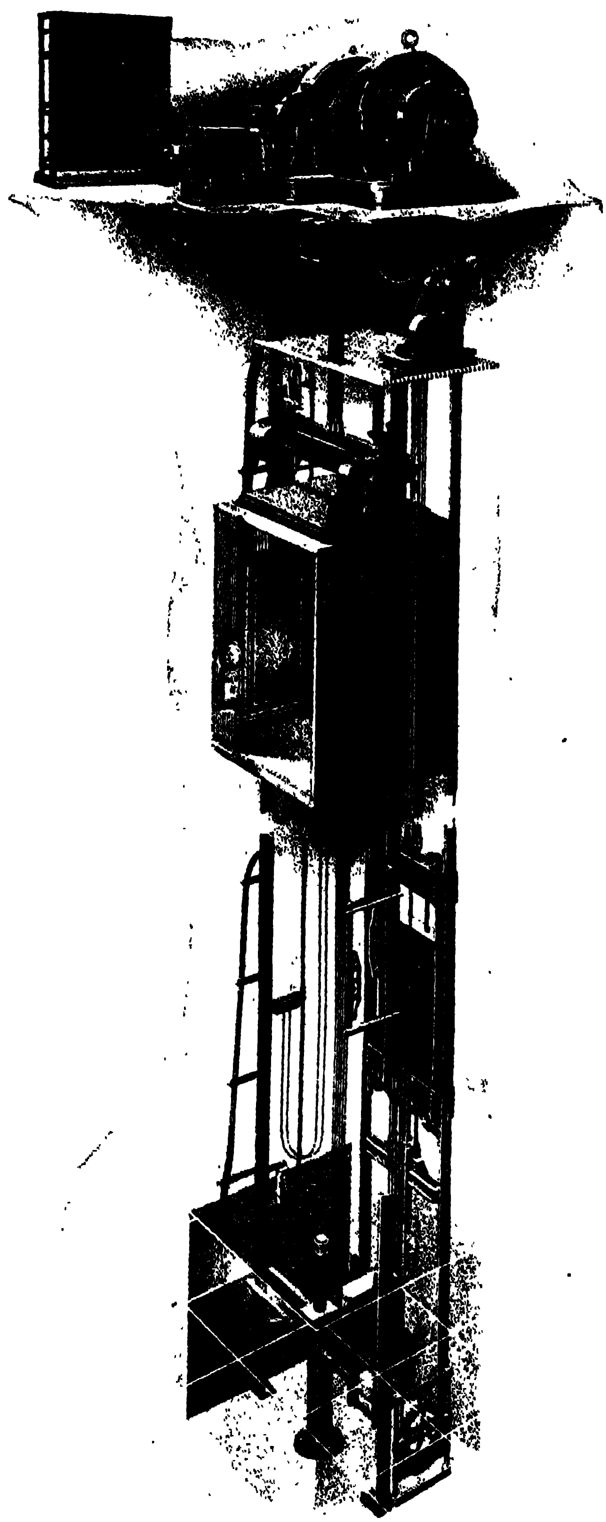


FIG. 517. GEARLESS PASSENGER ELEVATOR — DOUBLE WRAP TRACTION‡ (MICRO-DRIVE) 1 to 1 Roping — Machine Above

‡ Courtesy Otis Elevator Co.

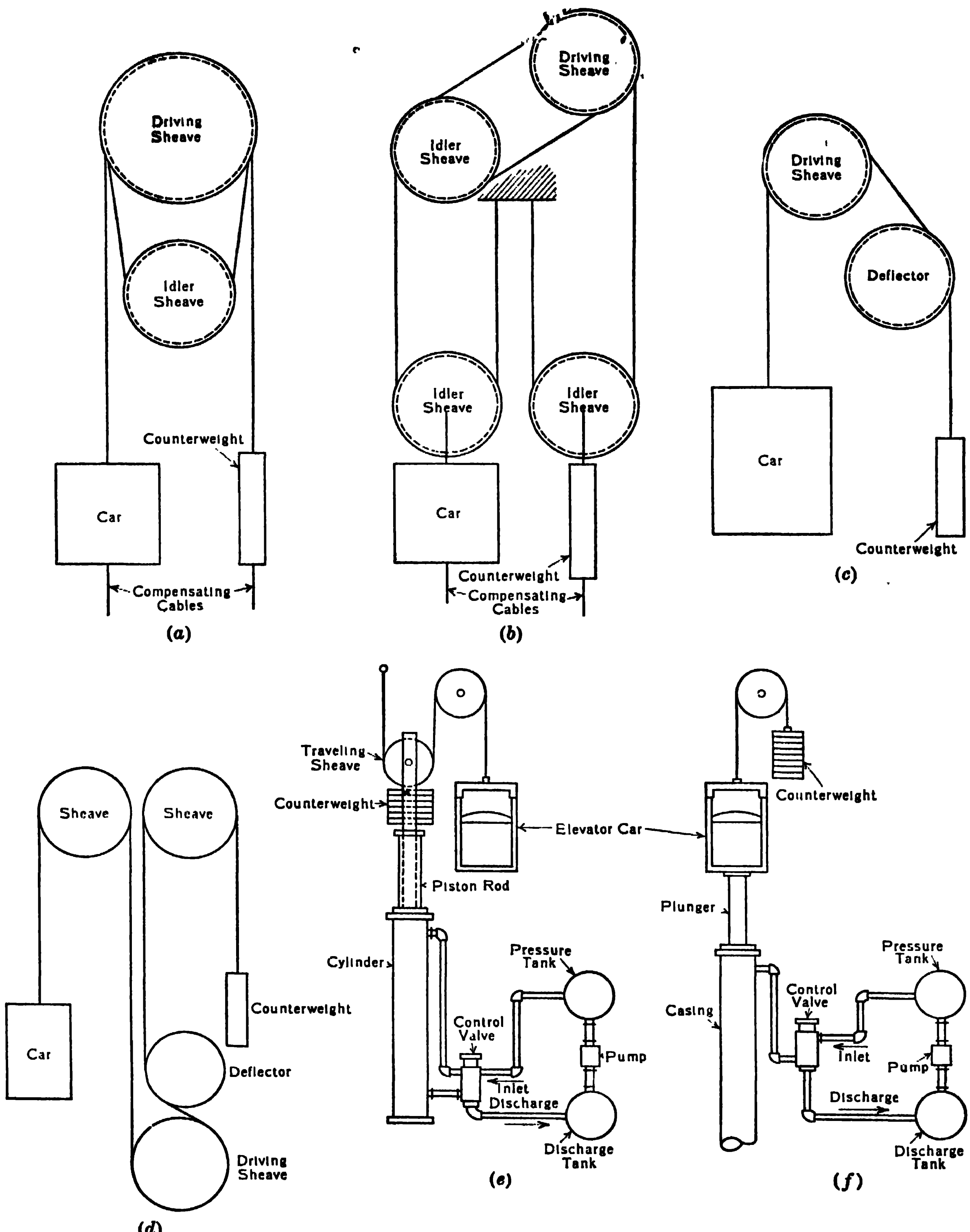


FIG. 518. ELEVATOR SCHEMES

(a) 1 to 1 roping, double wrap, gearless traction, machine overhead
(b) 2 to 1 roping, double wrap, gearless traction, machine overhead
(c) 1 to 1 roping, single wrap, geared traction, machine overhead
(d) 1 to 1 roping, single wrap, geared traction, machine below
(e) vertical counterweight cylinder — hydraulic
(f) direct acting car plunger — hydraulic

there are car openings shall be filled flush with the line of the shaftway or be beveled.

(*b*) The beveled plates shall extend from the edge of the projection to the vertical wall, and the beveled surfaces shall make an angle of not less than sixty degrees with the horizontal.

the particular manufacturer, but Fig. 520 may be used as a general guide, (*a*) showing dimensions for gearless passenger elevators and (*b*) for freight 1 : 1 elevators. Layouts will of course vary according to local conditions. In some cases when the en-

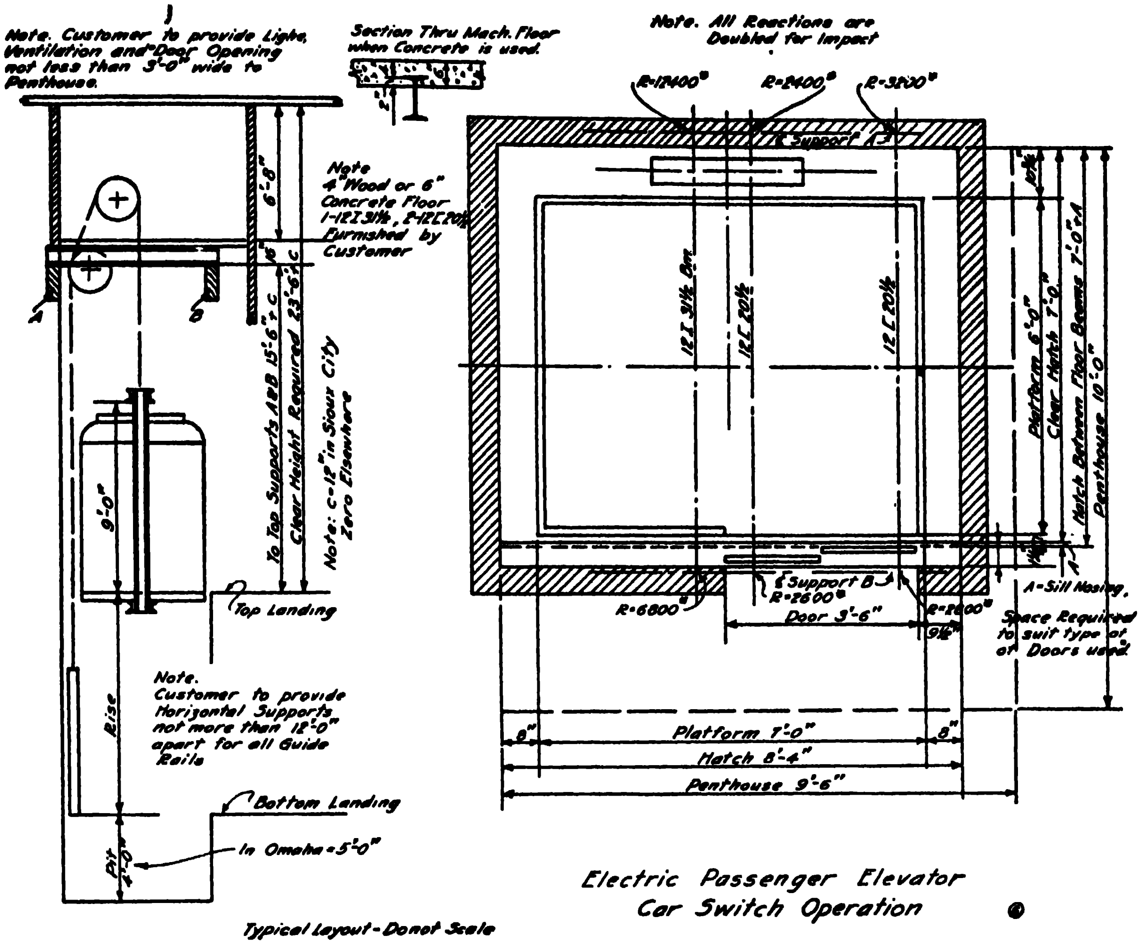


FIG. 519. PASSENGER ELEVATOR*

1 to 1 Roping — Single Wrap Traction — Car Switch Operation — Counterweight at Rear

Clearances Between Cars, Counterweights and Shaftways (*a*) There shall be a clearance of not less than three-fourths of an inch (¾") between cars and the shaftway enclosures and a clearance of not less than one inch (1") between cars and their counterweights.

(*b*) The clearance between the sill of car and the threshold of landing shall not be less than three-fourths of an inch (¾") nor more than one and one-half inches (1½").

(*c*) There shall be a clearance between the elevator counterweights and the shaftways of not less than three-fourths of an inch (¾").

These requirements may vary somewhat in different states but are typical of the usual cases.

In addition to the clearances, room must be provided for the counterweights, their guides and the car guides. These of course vary somewhat with

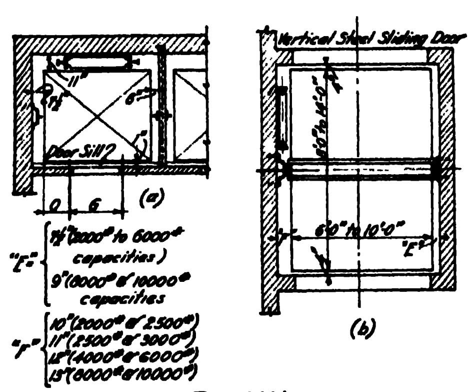


FIG. 520†

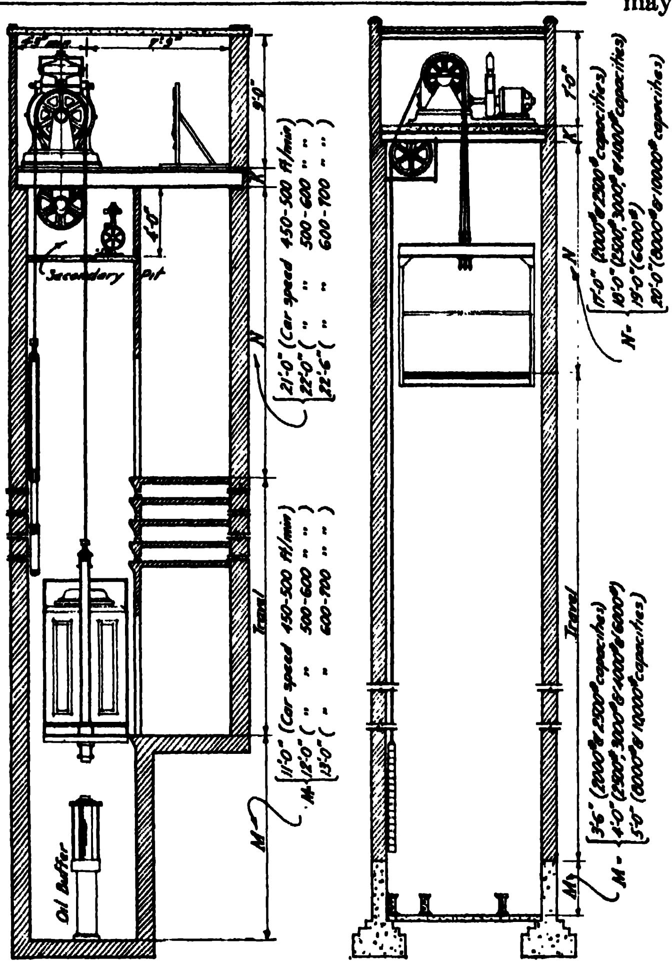
* Courtesy of Otis Elevator Co.

† Standards of the Kaestner and Hecht Co., Westinghouse — K & H Elevators. All clearances are in accordance with the A.S.M.E. Code.

trance at the first floor is on a side adjacent to the side where the entrances at the upper floors occur, corner guides* are used. This will affect counterweight locations. Similar considerations must be made when doors occur on opposite sides of a hatchway. Such variations affect the clearances.

Passenger car sizes are fixed according to the number of stories in the building and the number of cars to be used, and the car speeds according to the number of stories. They are also based somewhat on allowing 2□' for each passenger and 4□' for the operator. The following gives some average values:

Stories	Car Area	Car Load	Speed (Ft. per Min.)
8 to 13	25	1700	250 to 350
14 to 22	30	2000	350 to 600
23 to 30	40	3000	400 to 600



(a) FIG. 521† (b)

The car loads are more or less a function of the car area. The width of a car platform should be slightly greater than its depth. In Fig. 520, *O* represents the width required for the operator. This is usually 2'-3". The distance *G* is the gate opening required for efficient service. The following represents average dimensions:

Areas of Car Platforms	25□'	30□'	40□'
Width	6'-0"	6'-3"	7'-0"
Depth	4'-3"	4'-9"	5'-9"
G = gate opening	3'-9"	4'-0"	4'-9"

327. Vertical Clearances.

In order to provide clearances between the top of the elevator car and the sheaves, elevator machinery, governors and so on, vertical dimensions must be established above the top floor the elevator is to serve. These also allow for over-travel‡ or "over-run" (running car above top floor). One dimension given is from the top floor to the level at which the elevator machine is to be placed. This may be expressed as the distance to the bottom of the sheave beams (*N*), and then a dimension from this point to the finished floor, allowing for the depth of the sheave beams and the thickness of the floor (*K*). These are shown in Fig. 521 for one type of manufacture, those in (*a*) for gearless passenger elevators and those in (*b*) for freight 1 : 1 elevators. The value of *N* varies with the capacity for freight elevators and with the speed for passenger elevators. In some types of installations, a secondary pit is used for the governor and secondary sheaves, as shown in Fig. 521 (*a*). If the secondary pit is omitted, 1'-0" should be deducted from the values of *N* given. The dimension *K* is usually made a constant of 16", including an allowance for a 4" concrete slab.

With a value of *N* established, the relation of the machine room floor to the grade of the main roof may be determined by subtracting the top story height (top floor to finished roof surface). This will lead to what is required for access from the roof to the penthouse. Occasionally the elevator is extended to serve the roof (as for roof gardens, recreation spaces, etc.). This will raise the machine room floor accordingly.

Above the machine room floor, sufficient head-room must be furnished for the elevator machine, motor, controller, and so on. Such dimensions are shown in Fig. 521. Another consideration is the feature of placing or repairing the mechanism. In many cases a tackle and trolley are made a part of the in-

* Corner guides are not considered as efficient and should be avoided when possible, as side guides are stiffer.

† Kaestner and Hecht Co., Westinghouse — K. & H Elevators.

‡ This allowance varies with different manufacturers and types of cars, and also with the car speeds. In general, the over-travel at top is 5'-0" for car speeds of 450–500 ft. per min., 5'-6" for 500–600, and 6'-0" for 600–700, the allowance being generally made only for high-speed cars.

stallation. In such cases, 3′-0″ should be added to the dimensions given.

Two types of treatment are used at the sheave beams. One is to provide a solid fire-resisting platform (usually a concrete slab) surrounding the sheave beams and in which cable slots are provided. The other is to leave the sheave beams exposed and to supply other beams to support the machinery. Below the whole, a grating is provided (see Art. 338).

Another feature in connection with the hatchway is supplying the vertical depth of the pit. Figure 521 shows one set of standards. The pit depth must allow for the placing of buffers or bumpers. The minimum depth should be 3′-0″.* Where oil buffers or their equivalent are used the pits may be as deep as 15′-0″. The ordinary bumpers are so arranged that there is a clearance of 20″ between the floor of the pit and the car sling, when it rests upon the bumpers. The ordinary bumper is used for cars with a rated speed of less than 100 feet per minute, spring bumpers for speeds between 100 and 400 feet per minute and oil buffers for cars with speeds larger than 400 feet per minute. For high-speed cars, an allowance in the depth of the pit is made for over-travel † (running car by lower floor). The pit walls and slab are usually made of concrete and are generally a part of the foundation work.‡

Occasionally elevators are driven by machinery placed in the basement as illustrated in Fig. 522. This is often the case in alterations to existing buildings, and in structures where penthouses would interfere with the architectural treatment desired. Such construction usually requires supporting

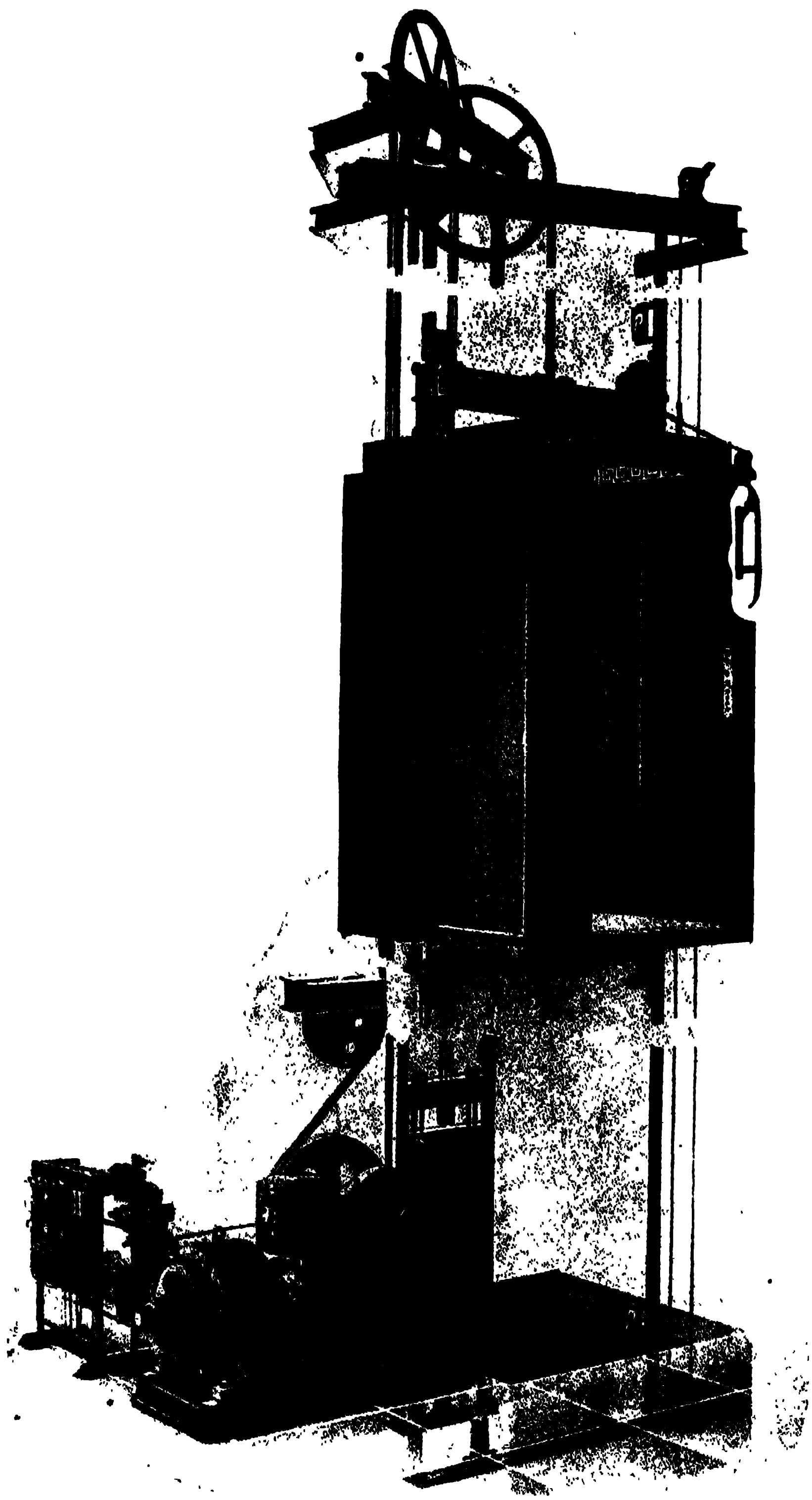


Fig. 522. Geared Passenger Elevator — Single Wrap Traction § 1 to 1 Roping — Machine Below

§ Courtesy of Otis Elevator Co.

* State regulations as well as city codes give various minimums.

† This allowance varies with different manufacturers and types of cars, and also with the car speeds. In general, the over-travel at bottom is 3′-6″ for car speeds of 450–500 ft. per min., 4′-6″ for 500–600, and 5′-6″ for 600–700.

‡ Refer to a text on concrete foundation work.

beams in the plane of the first floor to carry the deflector sheave only (Fig. 522), the remaining machinery being carried on the soil or on a footing. Sheave beam supports are required overhead, their loads generally being somewhat lighter than when an overhead drive is used. Such an arrangement requires that the machine-room be of fireproof construction. A penthouse of much less height may be used, — one which will give headroom for the sheaves only. The enclosures should extend through and above the roof about 3'-0", where the elevator serves the upper floor of the building, except in cases where a solid fire-resisting platform is located under the machinery, which is completely blocked by the sheaves, except for the cable slots. Where solid slabs are not used at the top of the shaftway a skylight should be provided in the roof of the penthouse, or windows may be located in the side walls of the penthouse enclosure (Art. 330).

328. Sources of Elevator Loads.*

The standard office building elevator car in most of our large buildings is one of 2500 pounds capacity. These passenger cars should be about 5 feet or a little less in depth and have a net area corresponding to about 80 pounds to the square foot. It has been found that large passenger cars, such as those used in the New York subways, are frequently loaded in service to 110 pounds to the square foot. It is desirable, and will be made mandatory by the Elevator Safety Codes, to provide a capacity varying with the car area about as follows:

25 sq. ft. — 75 lbs. per sq. ft.
30 sq. ft. — 79 lbs. per sq. ft.
35 sq. ft. — 82 lbs. per sq. ft.
50 sq. ft. — 89 lbs. per sq. ft.
100 sq. ft. — and above — 100 lbs. per sq. ft.

For purposes of general illustration, a L.L. on the area of the car platform of a freight elevator will be taken as 75#/□'. If a platform is 7'-0" × 8'-0", the total live load is then 7 × 8 × 75 = 4200#.

The modern counterweighted car is built with a steel suspension frame designed upon a factor of safety of 8 and a load equal to the maximum rated carrying capacity plus the weight of the unloaded car. Based upon calculations similar to the above, the car is given a maximum rated carrying capacity, or "duty." This will be called 4000# for this case. Cars are more or less standardized, such as 1500#, 2000#, 2500#, 3000# and 3500# for passenger elevators, and 2000#, 2500#, 3000#, 4000#, 6000#, 8000# and 10,000# for freight elevators. To the capacity of the car must be added its weight, including the safety frame, platform and cab (Fig. 523).

The **car safety frame** is made of structural steel securely bolted and riveted together, to which the hoisting ropes are attached by means of self-adjusting hitches, which provide means for equalizing the strains on the ropes and relieving them of undue twisting strains. Spring adjusting, self-aligning guide shoes are mounted on the car frame to secure smooth running. The **car platform** should be strongly built with a steel frame, securely braced to the safety frame, and should have a selected, well-seasoned wood floor fireproofed on the under side with sheet metal and the upper surface finished with tile or other floor covering.

The **counterweight** should be suitably counterbalanced for smooth and economical operation. The counterweight usually consists of heavy cast-iron sub-sections, contained in a structural steel frame provided with adjustable guide shoes. The weights should be properly consolidated by means of tie-rods.

The **cab** may be a standard of some manufacturer or may be specially designed by the architect.† It is usually made with panels of pressed steel (or bronze in special cases) with a baked enamel finish. It is usually fitted with ventilating grilles at the top, a folding gate, a suitable light fixture and operating device. The cab is generally provided with side and top emergency exit panels.

The weight of the car safety frame naturally varies considerably with the type of car. Here it will be assumed as 1090#. The cab or enclosure weight will also vary for different cases. Elevator companies of course have data for estimating such quantities. As an average, a value of about 16#/□' of platform area will approximate the weight for the ordinary height of cab. For purposes of illustration, this will be taken as 895#. The weight of car platform will vary with the size, kind of finish floor and so on, but a value of about 12#/□' represents an average. For a platform of 7'-0" × 8'-0", the weight is 7 × 8 × 12 = 672#, and allowing for fittings, it will be taken as 690#. This gives a total car weight of 1090 + 895 + 690 = 2675#.

A counterweight of a value equal to the weight of the unloaded car plus 40% of the capacity is usually provided.‡ For this case, the counterweight would be 2675# + 0.40 × 4000 = 2675 + 1600 = 4275#. The counterweight pull, as well as that from the car is exerted at the machine. The total active force, or the "**live load**," as it is called by the elevator companies, is then the sum of the car weight, capacity of the car, counterweight and weights of the cables (the guides being supported by the hatchway frame). If the cables weighed 155#, the total would be 2675 + 4000 + 4275 + 155 = 11,105#. The maximum "rope pull" would be the sum of the car weight, its capacity, and the weight of the cables, or 2675 + 4000 + 155 = 6830#. The size of the cables may be selected for this load. The "live load" could also be stated as the rope pull plus the counterweight, or 6830 + 4275 = 11,105#, as before.

Because of sudden startings and stoppings, elevator supports are subjected to considerable impact. **The usual practice is to allow 100% for impact.**§ Then the value of 11,105# should be doubled, or 2 × 11,105 = 22,210#, is the load considered on the supports. To this must be added the weight of the machine. Such weights are known from the manufacturer and vary according to the loads and speeds. Assume that this is established as 3300#. The total load at the machine to be considered is 22,210 + 3300 = 25,510#.¶

The next step is subdividing the load with respect to the plan of the hatchway. The center-lines of the platform in

* The numerical calculations given in this article are not to be considered as a definite procedure in obtaining elevator loads for any given case, and are given as a means of illustrating the general idea only. The establishment of such figures is the province of the elevator companies, and none but experienced elevator engineers should make such calculations.

† See Volume I.

‡ It would of course be unreasonable to counterweight for the full live load in the car, as this would not give efficient operation. A value of 40% is the result of experience. The counterweight should not be less than the weight of the unloaded car in order to descend, yet it must compensate the weight of the car and an average load.

§ Some engineers consider that the supporting beams should be designed for this allowance, but design the supporting columns only for the actual loads. This variation is not recommended.

¶ The total weight may then be expressed as 2 (car weight + 1.4 × load in car) + weight of machine for the case when the machine is overhead. For a case of the **machine below**, it is usually 4 × car weight + 2.8 × load in car + weight of machine, as the suspended weight must be added to the equivalent down-pull on the ropes, on the machine side of the overhead sheaves.

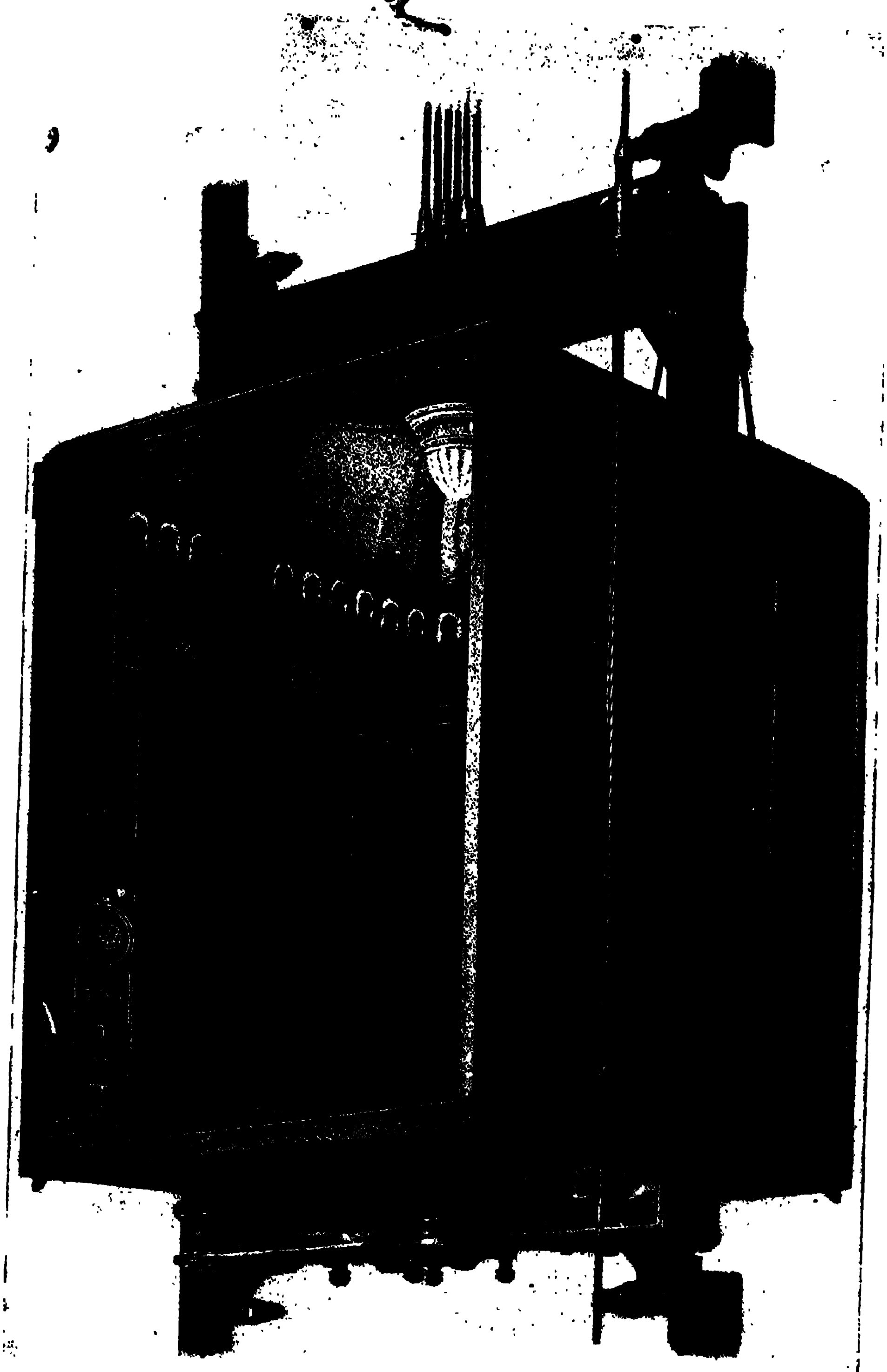


FIG. 523. OTIS SAFETY FRAME AND CAR*

* Courtesy Otis Elevator Co.

both directions are designated by X–X and Y–Y respectively, in Fig. 524 (*a*), or the center of the car is at L. The distance from the latter point to the center of the counterweight may be determined, as LK. The diameter of the driving sheave is established by the design. This is usually 30″ to 36″. Since the rim of this sheave should be over the center of the car to pick up the cables, the center of the sheave is fixed with relation to the car center, as point M. With KL known and the diameters of the driving sheave and deflector sheave fixed, the relation of the sheaves in a vertical direction may be established. In this case, the counterweight is placed in the rear of the hatchway and centered on the center line of the platform,—at point K on line Y–Y. This allows the sheave beams to be placed parallel to a side of the car. If the counterweight has to be placed to one side in the rear or on one side or the other of the car, due to the layout, then the sheave beams will have to be placed diagonally, as in Fig. 514.

While the total load discussed above theoretically acts at M in Fig. 524, the elevator machine, including the driving sheaves and motor, is usually built into a unit (Fig. 522) and distributes the load along the line W–W in a lateral direction. Elevator companies have tables which give the distances and proportions of loads to the seating points for their various types of machines, such as the loads at A, B, and C in Fig. 524. A and B are approximately 1′-0″ apart and C is approximately 1′-9″ from B. The point M (center of driving sheave) is usually about ⅔ the way between A and B. Assume the loads at A, B, and C are 13,350, 7810, and 3380 respectively, or 13,950 + 8125 + 3435 = 25,510#, the value obtained above. The loads at A, B, and C are then transferred to the supports by the sheave beams on the lines R–R, S–S, and T–T respectively. Figure 524 (*b*) shows a beam loading sketch in which P is the load at A, B, and C in each case.

Each sheave beam is designed in a typical manner. On the line R–R, the bending moment is

$$M_c = \frac{P \cdot a \cdot b}{L} = \frac{13{,}950 \times 3.79 \times 5.96}{9.75}$$

$$= 32{,}400'\#.$$

Assume beam weighs 31.8# per ft. The moment due to its weight is

$$M_w = \frac{31.8 \times (9.75)^2}{8} = 380'\#.$$

The total moment is 32,400 + 380 = 32,780′#. A working stress of 12,000#/□″ is generally recommended. Then using

$$M = \frac{s \cdot I}{c} \quad \text{or} \quad \frac{I}{c} = \frac{M}{s}, \quad \frac{I}{c} = \frac{32{,}780 \times 12}{12{,}000} = 32.78''^3$$

$$\text{Use } 12\ \text{I}\ 31.8 \left(\frac{I}{c} = 36.0''^3\right)$$

The other sheave beams are designed similarly. They are generally kept to 12″ or 15″ depths (minimum weights if larger sizes are not required), in order to keep the 16″ dimension (previously discussed) from the bottom of the sheave beams to the finished machine-room floor.

The proportions of load which are in turn brought to the sheave beam supports may now be calculated. On line R–R, R_1 in Fig. 524 (*b*), (the load at d) is

$$\frac{3.79}{9.75} \times 13{,}950 = 5420\#$$

plus one-half the beam weight, or ½ × 9.75 × 31.8 = 155#. The load at d is then 5420 + 155 = 5575#. The load at a

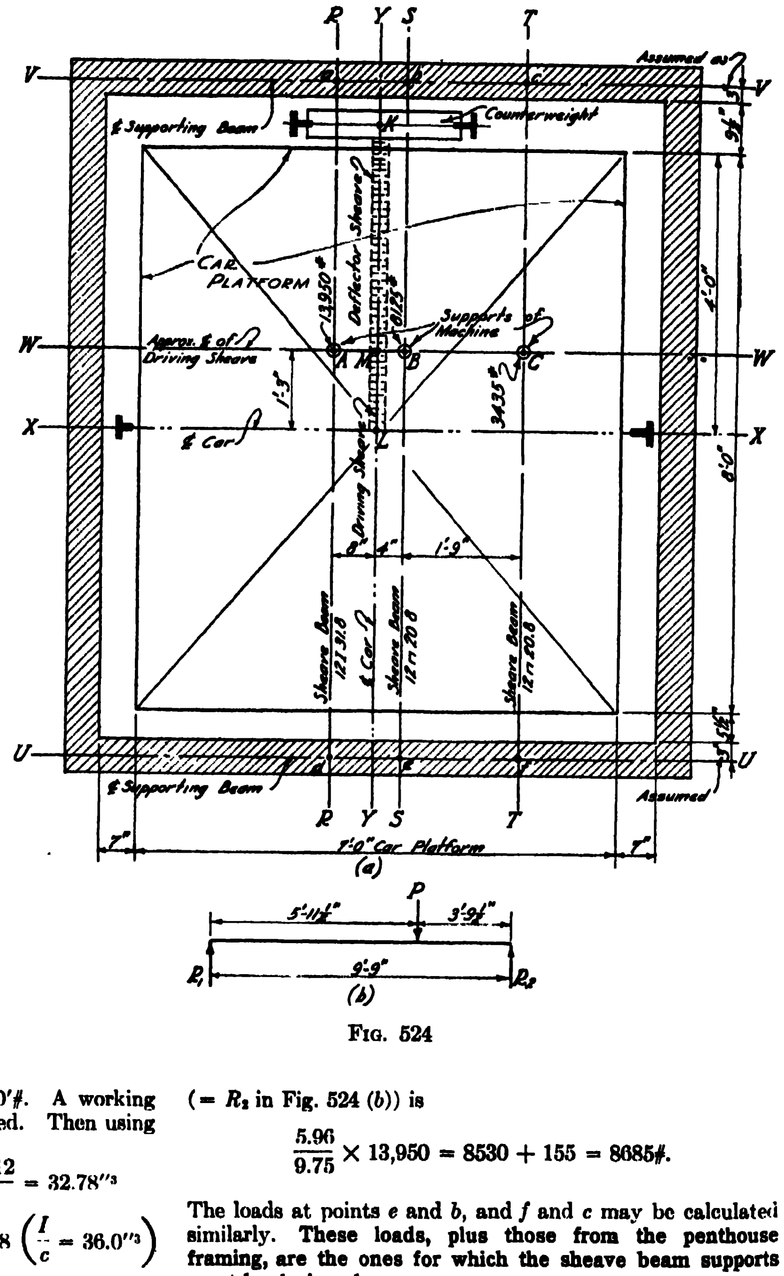


Fig. 524

(= R_2 in Fig. 524 (*b*)) is

$$\frac{5.96}{9.75} \times 13{,}950 = 8530 + 155 = 8685\#.$$

The loads at points e and b, and f and c may be calculated similarly. These loads, plus those from the penthouse framing, are the ones for which the sheave beam supports must be designed.

The loads should be established by the elevator company, but for purposes of preliminary investigations the following data may be used as a guide.

TABLE 81

APPROXIMATE TOTAL LOADS ON SUPPORTS FOR PASSENGER ELEVATORS

GEARLESS 1 : 1 ELEVATORS*

These loads have been doubled for impact

Speed of car (Ft. per min.)	Capacity, pounds 1500	2000	2500	3000	3500
50	17,900	19,200	24,800	26,000	35,400
100	17,900	22,900	32,900	34,200	35,400
150	17,900	25,000	32,900	34,200	38,800
200	17,900	25,000	32,900	36,900	38,800
250	25,000	25,800	35,800	36,900	46,200
300	25,000	25,800	35,800	36,900	46,200
350	32,100	34,200	35,800	44,100	46,200
400	32,100	34,200	35,800	44,100	46,200
450	32,100	34,200	35,800	44,100	46,200
500		56,200	57,500	58,800	61,000
550		56,200	57,500	58,800	61,000
600		56,200	57,500	58,800	61,000
700		56,200	57,500	58,800	61,000

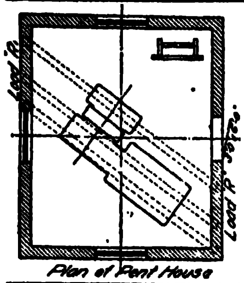
* Kaestner and Hecht Co., Westinghouse — K & H Elevators.

TABLE 82

APPROXIMATE LOADS ON SUPPORTS†

FREIGHT 1 : 1 ELEVATORS

These loads have been doubled for impact



Capacity	Speed	R	R_1
2,000	50–100	8,500	13,000
2,500	50–100	10,500	14,000
2,500	150–200	12,500	18,000
3,000	100–200	14,000	20,000
4,000	100–125	19,000	28,000
6,000	100–150	22,000	31,000
8,000	100–150	26,500	35,000
10,000	100–125	28,000	37,500

† Kaestner and Hecht Co., Westinghouse — K & H Elevators.

329. Procedure by Structural Engineer.

When the structural designer is ready to proceed with the design of the supports which carry the sheave beams, his first step is to study the architectural drawings to get a general idea of the pro-

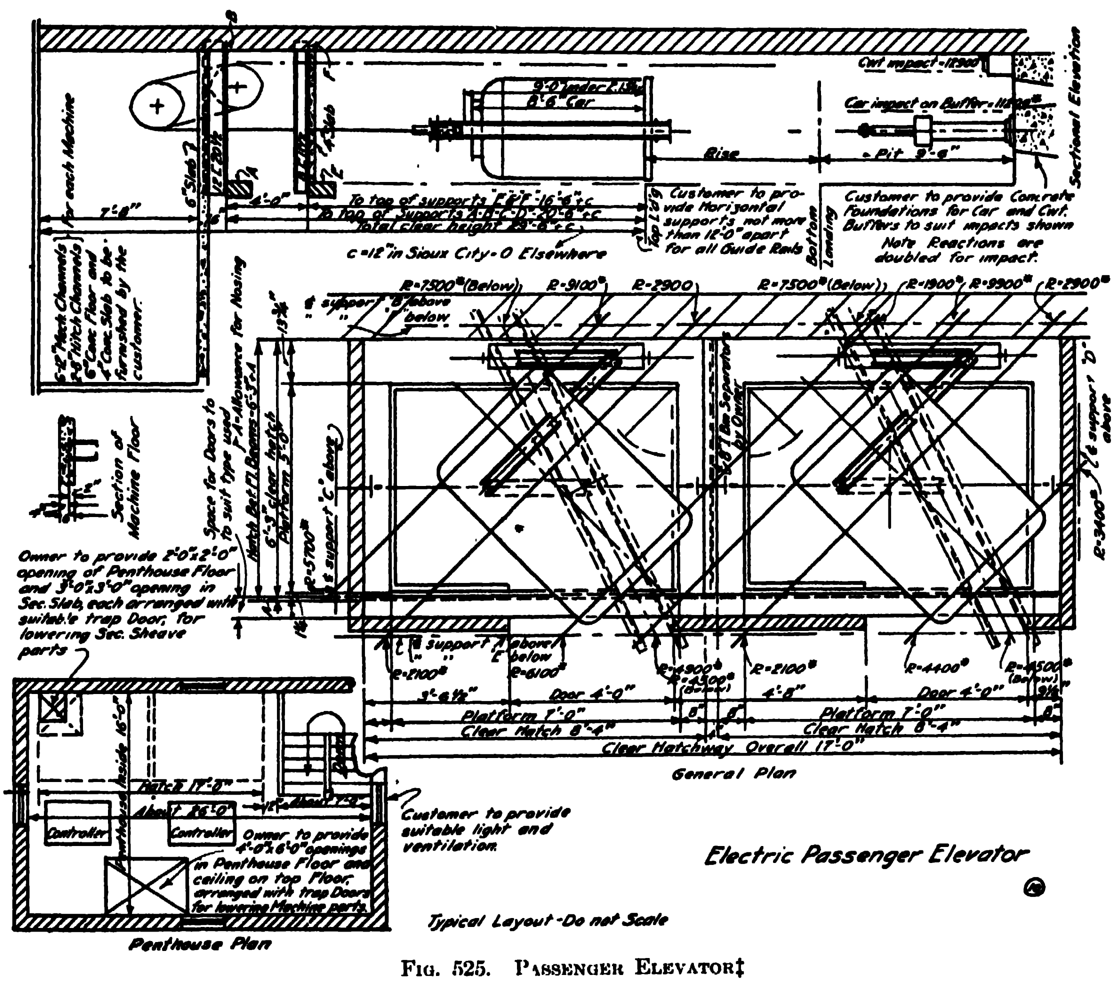


Fig. 525. Passenger Elevator‡

2 to 1 Roping—Double Wrap Gearless Traction—Car Switch Operation — Counterweight at Rear — Oil Buffers

‡ Courtesy of Otis Elevator Co.

posed arrangement. This will require a reference to the typical plans for the hatchway, the roof plan, penthouse roof plan and sections.

The first step will be to decide what the nature of the supports is to be. The general architectural treatment will influence this. If the penthouse walls are to be of brick or concrete, the sheave beams may be supported directly on the walls, using the the sheave beams, then one method is to supply bearing plates in the wall for them. Some engineers prefer to embed a grillage beam, usually a piece of 8 BH 32, in the wall to seat the sheave beams, particularly if there are quite a few of them. This allows the sheave beams to be placed in any position near that planned for, or in the case of diagonal sheave beams, it allows them to be placed at a

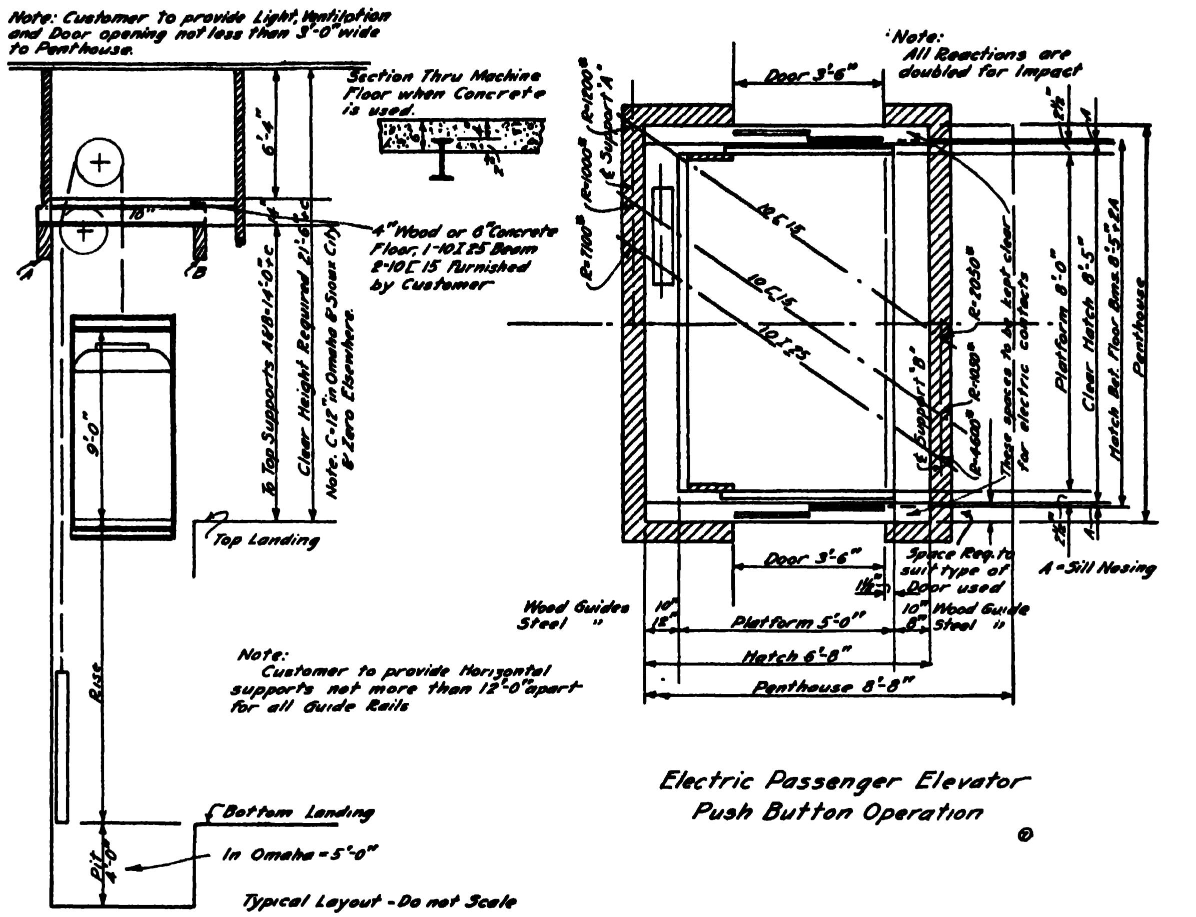


FIG. 526. PASSENGER ELEVATOR (HOSPITALS)*
1 to 1 Roping — Single Wrap Traction — Push Button Operation — Counterweight at Side

latter as bearing walls. These walls may be supported by beams at the plane of the roof, or occasionally, in buildings of small height, particularly in manufacturing or warehouse types, the enclosure walls may be made bearing walls continuous to the pit and carried by their own footings. Usually, however, the plan size of the penthouse is larger than that of the hatchway, to provide access to the machines and to provide working space as in Fig. 525. This necessitates a framing arrangement at the roof plane. If bearing walls are used to support different angle if it becomes necessary during the elevator installation.

Penthouses are more often made of 6" terra cotta blocks or other partition material of similar thickness, and flashed and counterflashed at the roof level and then copper covered. Such walls do not provide bearing for the sheave beams and it is necessary to use a structural frame for this purpose, strutting down at the corners to the roof frame, or if convenient, carrying up four columns of the main building to provide supports.

When the type of supports is decided upon, the

* Courtesy Otis Elevator Co.

designer refers to the elevator drawing, furnished by the elevator company. Plate 41 gives a typical illustration of this kind for a freight elevator. Figure 526 shows a similar condensed layout for a passenger elevator. The engineer should give it more than superficial study, in order to make sure that he fully understands the data. There are a number of features which he should note, among which the most important are:*

(1) Type of elevator.
(2) Whether elevator machinery is to be located overhead or in basement.
(3) Size of hatchway at roof plane.
(4) Height of machine-room floor above roof.
(5) Distance from finished machine-room floor to bottom of sheave beams.
(6) Plan-size of penthouse.
(7) Height of penthouse roof above machine-room floor.
(8) Whether crane beams are to be provided or not.
(9) Whether a solid machine-room floor is to be used, or simply open sheave beams and a grating beneath.
(10) Whether a secondary pit below the sheave beams is shown or not.
(11) The points of application of the loads from the sheave beams.
(12) The values of the loads from the sheave beams.
(13) Whether the loads given on the elevator drawing include an allowance for impact or not.

The last note (13) is a very important one. Some companies give the loads allowing for impact, while others give the loads due to direct weights and note that such loads should be doubled for impact.†

The design of beams acting as sheave beam supports involves no new features beyond those previously discussed except the features of working stresses and lateral support. The following notes are commonly given on elevator drawings:

The following fibre stresses are recommended:

Steel	**12,000# per sq. in.**
Yellow pine or oak	**1,500# per sq. in.**
White pine or spruce	**1,200# per sq. in.**

To insure proper stiffness for supporting beams, the span of any beam should not exceed 12 times its depth. Supports marked thus (identifying mark) are to be furnished by Owner.‡

* Considerations for the frame around the hatchway at the various floors must also be made (Art. 331). Data relative to the pit must also be noted in connection with the basement and foundation arrangements.

† The latter method is not quite as satisfactory, as the loads include the proportional parts of the weight of the elevator machine, which theoretically by itself does not need an impact allowance, as it is a static load. This method of doubling the loads given for impact gives slightly larger values and requires a little more material in the supporting beams.

The general contractor is often required to sign the elevator drawing thus approving the arrangement and he must guarantee usually that the hatchway sizes will be within 1″ of the figures shown.

330. Penthouses and Machine-rooms.

The plan size of the penthouse is generally larger than that of the hatchway, and is extended on one or two sides beyond the elevator shaft opening in order to provide access around the machinery. The size depends upon the size of the elevator machine and the counterweight. Space is thus provided for the accommodation of one or more controllers, and for installing or repairing the apparatus.

If a solid machine-room floor is to be used, it is generally a concrete slab (6″ common,— of course depending upon the span). It should be designed for a **live load of 250#/□′**, as such a load is often realized. Controllers weigh from 1000# to 2000# and occupy a relatively small area, and heavy machines, if taken off the sheave beams for repair, would approach such a load on their bases. The structural engineer does not attempt to locate the slots for cables, but usually puts a note on the framing plan that " general contractor shall provide slots in slab as directed by the elevator contractor." Openings for trap doors, as called for on the elevator drawing, should be detailed on the plan, with extra rods at the sides of the openings and diagonal, short rods at the corners, to act as " hair pins." Such openings sometimes occur in machine-room floors and also at roof planes (Fig. 525). If a grating is used below the overhead work as a working floor, instead of a slab at the sheave beam level, it should be strong enough to support a live load of at least 75#/□′ (Art. 338).

The roof of the penthouse is generally made of a concrete slab with suitable roof covering. Either a single slab or a series of slabs and beams may be used, depending upon the spans. If a tackle and hoist is to be provided for, a crane (trolley) beam (Art. 372) should be introduced.

When a solid slab is not used at the top of the hatchway, a skylight or windows in the sidewalls of the enclosure should be employed. There are State codes regulating such features. In general, the glass area of such skylights or windows should not be less than 50% of the cross-sectional area of the shafting nor less than 3 sq. ft. The glass in the skylight or windows should be plain glass instead of wired glass so that it may be broken in case of fire. If wired glass is used, the skylight or sash should be pivoted so that it may be opened to provide

‡ The sheave beams themselves are a part of the elevator contract, as well as the guides and their means of attachment. Supports for the guides must be provided.

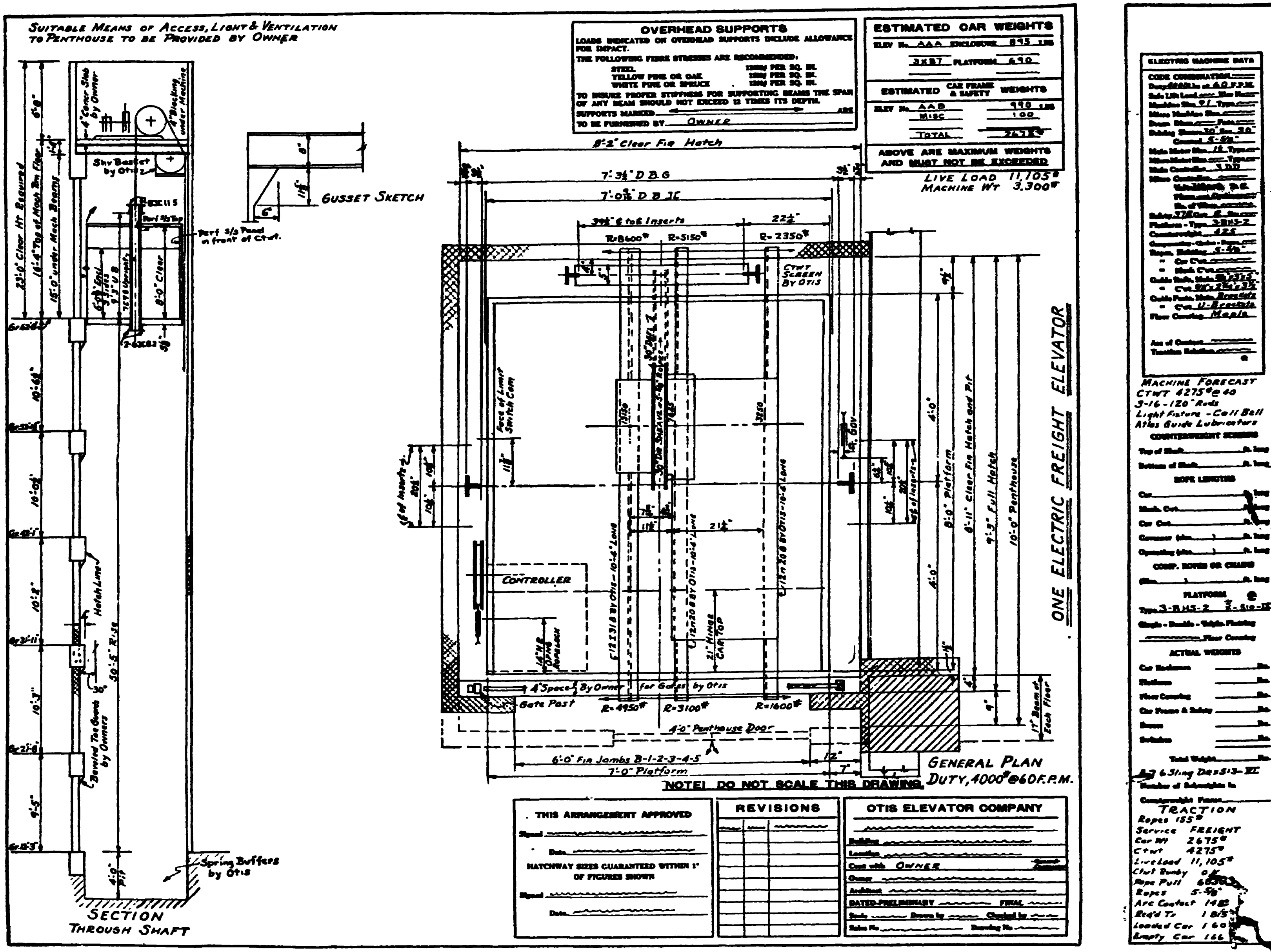


PLATE 41 TYPICAL ELEVATOR DRAWING

Courtesy of the Otis Elevator Company

100% ventilation. The plain glass of skylights should be protected by wire screens of about 1" and not less than No. 12 gauge wire, placed about 6" above the skylight. The vertical panes of plain glass should be protected by gratings made of $\frac{5}{8}''\phi$ rods about 10" o.c., and fastened into the masonry walls. A door opening should be provided in the enclosure large enough to move the machines in or out. The door should be a fire-door conforming with the requirements of the Underwriters Laboratories, Inc.

331. Hatchways at Floors.

The enclosures of the hatchways at the various floors are of masonry or specially designed grilles and are a part of the architectural details. The walls may be made of brick, hollow tile, wire lath and cement plaster, wired glass set in metal frames, or open metal grilles.* The doors are usually of pressed steel with baked enamel or of bronze. They are made of two-speed type, or center-opening, and usually are self-closing.

SPECIFICATION CLAUSES

Elevator Entrances

Quantity Six (6) plate bronze passenger enclosures required, serving three cars at the first and second floors, each approximately 3'-6" wide × 7'-0" high and constructed in accordance with a set of two-speed doors sliding behind a wall partition. Forty-two (42) typical steel passenger enclosures required, each approximately 3'-6" wide × 7'-0" high, serving three cars at the third to roof floors inclusive, and constructed in accordance with a set of two-speed doors sliding behind a wall partition. Fifteen (15) steel service enclosures required, service elevator No. 2 at the first to fifteenth floors inclusive, each approximately 4'-0" wide × 7'-0" high and constructed in accordance with a set of two-speed doors sliding behind a wall partition. Seventeen (17) Peelle doors required, each approximately 6'-6" wide × 7'-0" high and constructed in accordance with the following specifications.

Contract for elevator entrances shall be let as a unit to one manufacturer, who will assume undivided responsibility for the complete installation.

Scope of Work The following specifications are intended to provide for a complete elevator entrance installation, including all of the metal parts necessary, consisting of toe guards, sills, jambs, trim, header, hardware, cover plate for hardware, door panels, also the glass, glazing, finish, and rubber bumpers; all as described below.

Materials All of the materials shall be the best of their respective kinds. All steel shall be 4-pass cold-rolled furniture stock of generally 12 gauge.

Workmanship The workmanship throughout shall be of the very highest grade. All work shall be done by mechanics and craftsmen who are experienced in the making of elevator entrances of the best type. All welds shall extend solidly clear through the steel, developing the full strength of the members welded, and the excess welding metal ground off smooth so that the weld cannot be seen. No brazing or soldering will be permitted.

Sills Sills shall be $\frac{3}{4}''$ thick of cast bronze for the bronze enclosures and of cast iron for the steel enclosures, with guide grooves for doors machine planed in the solid casting; non-slip, fluted and stippled safety treads in doorways. The cast-iron sills for the seventeen Peelle doors shall be included.

Sill Supports Steel brackets or necessary angle supports shall be provided to maintain proper relation to the floor construction.

Toe Guards Toe guards shall be of 12 gauge formed bronze for the bronze enclosures and 12 gauge formed steel for the steel enclosures, as per detail.

Jambs and Header The jamb and header shall be of 12 gauge formed bronze for the bronze enclosures and 12 gauge formed steel for the steel enclosures to suit the size of partition as indicated on architect's drawing and as per detail, with recesses and pockets where required.

Supporting Struts Supporting struts shall be provided to support the entrances, tracks, and header independently of the surrounding walls. Struts shall extend from floor beam to ceiling beam.

Trim Marble trim shall be supplied for the bronze enclosures, same to be furnished and set by marble contractor. For the steel passenger and service enclosures trim shall be of No. 16 gauge steel formed to detail with cast-iron plinth blocks. Trim shall be attached to the jambs with concealed fastenings; no bolts or screw heads being visible.

Hanger Housing The hanger housing shall be of steel plate entirely enclosing tracks and hangers; hinged and latched cover plate on shaft side to provide easy access to mechanism for oiling, etc. The finish of exposed parts shall be the same as remainder of entrances.

Hardware Hangers shall be of silent ball bearing type as approved by the Architect, to suit type of door layout indicated on drawings. Provide complete set of large size pure gum rubber bumpers. All locks and semi-automatic door closing devices shall be furnished and installed by the elevator contractor. This contractor, however, shall supply the necessary structural supports to receive same.

Door Panels The door panels for the plate bronze entrances shall be made in strict accordance with the design indicated, constructed of seamless bronze stiles and head $1\frac{1}{2}'' \times 1\frac{1}{4}''$, and $2'' \times 1\frac{1}{4}''$ of closed rectangles. Doors shall be made in solid welds throughout. Sash for mirrors shall consist of drawn bronze molding mitred and welded, with removable glass stops on shaft side.

Base panel shall be double-faced, asbestos filled, and solidly welded to door frame. Provide on bottom of door panels rectangular guides pivoted to accurately fit machine planed grooves in sills. The door panels for the steel entrances except entrances having Peelle doors shall be made in strict accordance with the design indicated, constructed of seamless steel stiles and head $2\frac{1}{2}'' \times 1\frac{1}{4}''$, and $2'' \times 1\frac{1}{4}''$ of closed rectangles. Doors shall be made in solid

* The local building code should be consulted in regard to such work.

welds throughout. Sash shall consist of a steel molding mitred and welded. Panel shall be doubled faced, asbestos filled, and securely fastened to door frame. Provide on bottom of door panels rectangular guides pivoted to accurately fit machine planed grooves in sills.

Mirrors Mirrors for the bronze entrances shall be beveled and same shall be backed up with bronze plates.

Finish The service enclosures shall have a gun metal Berlin black finish consisting of pickling, one coat filler baked and rubbed; one coat size; one coat gloss; and one coat Berlin black baked and rubbed. All of the typical steel passenger enclosures shall have a solid color baked enamel finish equal to seven coat work, each coat shall be rubbed down smooth with powdered pumice. Color shall be selected by the Architect. The bronze entrances shall have a natural brushed bronze finish.

Protection All work shall be carefully boxed and crated immediately after finish is dry and material is ready for shipment. Frame protection shall remain on until after plastering is finished and building is broom clean, and then shall be taken down by this contractor and removed from the building.

Erection Erection preferably shall be started at the top of shaft and worked down. All sills shall be firmly bolted and set absolutely level and plumb with the edge of the car platform so that there will be no variation in the distance between the edge of all of the sills and the edge of the car platform. Sill and jamb extension angles shall be bolted to the structural frame of the building. No bent clip fastenings will be permitted. The jambs and every part of the front shall be set absolutely plumb in all directions. The guide grooves in the sills shall be thoroughly cleaned of all dirt and grit and lightly coated with pure flake graphite. The hangers and locks shall be properly adjusted so that doors hang perfectly plumb with the jambs. All mechanism shall be carefully cleaned and greased. All metal work of enclosures shall be cleaned and the entire installation left in first class condition.

Inspection About thirty days after building is opened for occupancy all work shall be thoroughly inspected and the mechanism oiled and adjusted, and the doors plumbed at any points found necessary, as a part of this specification, and without extra compensation.

Peelle Doors The contractor shall furnish and erect complete and in good working order, Peelle doors as hereinafter specified in openings in large service elevator shaft in the building.

Total number of doors seventeen (17). Approximate sizes 6'-0" wide × 7'-0" high. These doors shall be the Peelle Patented Truckable Counterbalance Fireproof Doors, having standard galvanized panels set and bolted into rigid steel angle frames, to be equipped with the latest improved hangers, ⅝" turnbuckle rods, steel chains with large double radial ball bearing pulleys and operated in heavy steel anti-friction guides.

Each door shall be equipped with the Peelle patented trucking sill, which when the door is open, rests upon solid adjustable stops riveted to the guide rails.

These doors shall be manually operated and provided with web strap closers within easy reach of the elevator operator. Each door shall be provided with a type B Peelle electric contact, contacts to be wired in series and wires brought to the elevator control board, but physical connection to be made by the elevator company. There shall also be placed underneath the hanger bars at each side of the elevator car, bar latches operating from cams placed by the contractor on the elevator car to permit the doors to be opened from the loft side only when the car is at the landing level. Each door shall have in the upper section a 12" × 12" vision panel glazed with clear wire glass.

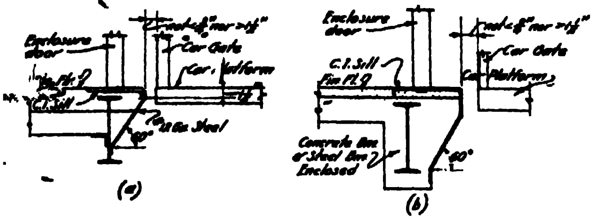
All shall be completely erected in place and left in perfect running order and guaranteed for two years against defects. The contractor shall guarantee this installation to meet the requirements of all Boards having jurisdiction.

Where there is a bank of two or more elevators, the space between adjacent cars is generally left open, that is, without a dividing partition. However, a beam is provided at the plane of the floor frame, running between the hatchways. This is usually made an 8 I 18.4 and need not be fireproofed, as it will serve more easily as a point of attachment for the elevator guides.* Such supports should not be farther apart vertically than 12'-0", and in a large story height, it may be necessary to introduce an intermediate beam for this purpose. The car and the counterweight are guided by specially milled steel tees (furnished by the elevator contractor), which should be securely fastened to the framing of the hatchway by iron brackets. The guides should be of specially heavy sections with planed surfaces, and should have their ends tongued and grooved to form matched joints, thereby insuring perfect alignment and smooth running. The shaft structural steel should be plumbed to a guaranteed vertical alignment of 1" in 20 stories and 1½" in more than 20 stories in order to insure the efficiency of the guides and to maintain clearances. Where the counterweight is in the same shaft, it should be protected by a screen.

At the sill of the door to the elevator shaft, and at all beams and other fixed construction projecting into the shaftway on car opening sides, the beam underneath should be built out to form a toe-guard (see Pl. 41). This is sometimes done by forming a beveled, metal plate of No. 12 gauge, which is attached to the beam, as illustrated in Fig. 527 (also see specification, Art. 326). If the beam in question is a concrete beam or a steel beam fireproofed, the toe-guard may be formed in concrete. The structural beam at the door opening should be

* Any beams serving as stiffeners or ties on column center-lines should always be fireproofed, however.

so located that its flange or fireproofing will not encroach on the clearance called for by the elevator drawing. A cast-iron or bronze saddle usually forms the threshold, and the top of the beam should



(a) (b)

Fig. 527

be depressed enough to allow proper bedding of such sills. The enclosure openings are usually finished with channel or angle frames with mitred corners. Special angles may also be necessary for hanging the doors. Such angles and channels, together with the sills, should be a part of the elevator door contract (usually a separate contract with a door company and not with the elevator company).

332. Dumbwaiters and Lifts.

In various types of buildings, minor lifts such as dumbwaiters, book lifts (Art. 340) and ash lifts, are a part of the equipment. If not operated by hand, the machinery is usually located in the basement, and is ordinarily light enough to be carried by the basement slab. Similar data from companies specializing in such work and who hold the patent rights on their own type of equipment may be obtained as to sizes of cars, hatchways, clearances, and loads to be carried by the structural frame.

CHAPTER 29

SPECIAL CONSTRUCTION

333. Miscellaneous Steel and Iron Work.*

Viewing the construction of a building as a whole, there are a number of instances in various types of structures where structural steel shapes and other related materials are used. These are commonly grouped under a heading in the specifications as "Miscellaneous Steel and Iron Work," and in the trade, they are called "light iron." Some structural companies have an ornamental iron department and take on such work as a part of their contract. There are also separate companies which make a specialty of such details.

The detail design of the "light-iron work" is generally not a part of the preparation of the structural framing plans, but every structural engineer should be familiar with the general features of such details, and the way in which they affect the framing. The engineer may be called upon to consult with the architect in regard to such work, or to check shop drawings submitted by a contractor for approval. The following is a partial list of possible instances of such work:

(1) **Anchors** for stone work, terra cotta, brick work, block partitions, furring, and the like.
(2) **Angle corner guards,** to protect concrete or concrete fireproofed columns from abrasion, such as in boiler rooms, shipping rooms, coal rooms, auto shops, and the like. These are commonly $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{16}$, 4'-6" long, tied in with $1'' \times \frac{1}{8}''$ anchors, 8" long, spaced 18" o.c., staggered.
(3) **Angle frames,** for louvres, grilles, hose cabinets, or other miscellaneous supports. These are usually of $3 \times 3 \times \frac{1}{4}$ angles with mitred joints at the corners, anchored into the masonry.
(4) **Ash hoists** and supports.
(5) **Awning box supports** (Art. 334).
(6) **Bulkhead framing** in storefronts.
(7) **Cast-iron cleanout doors** and **angle frames,** in chimneys and stacks.
(8) **Cast-iron cleanout boxes** and frames, for traps, etc.
(9) **Cast-iron grilles.**
(10) **Cast-iron pilasters,** in storefronts.
(11) **Cast-iron sills,** at elevator entrances, basement door entrances, and so on, usually $\frac{3}{4}''$ thick with $\frac{3}{16}''$ corrugations in surface, and $1\frac{1}{2}''$ grout holes at ends and middle.
(12) **Coal chutes** and angle frames.
(13) **Coal holes,** frames and covers.
(14) **Dampers for fireplaces.**
(15) **Door bucks, for elevator enclosures and fire door openings, usually of steel channels, rolled or pressed sections.**
(16) **Elevator pans.**
(17) **Flagpoles** (Art. 336).
(18) **Furring lintels, usually not a part of the regular structural lintels, — commonly $2 \times 2 \times \frac{1}{4}$** (Art. 293).
(19) **Gratings, for areas, boiler rooms, and so on** (Art. 338).
(20) **Hose cabinets, usually #16 gauge with #12 gauge** trim.
(21) **Ladders** (Art. 311).
(22) **Louvres** and angle frames.
(24) **Mantel lintels** — fireplaces.
(25) **Marquises** (Art. 335).
(26) **Pipe railings.**
(27) **Porch rails** (Art. 338).
(28) **Rolling or folding shutters,** for boiler rooms or shipping openings.
(29) **Stacks,** steel interior with **hoods** (Art. 343), either self-supporting, or with steel channel supports.
(30) **Stairs** (Chap. 26), steel sub-tread and riser, and **safety treads** for concrete stairs.
(31) **Sidewalk vault framing** (Art. 337).
(32) **Sky-signs** (Art. 346).
(33) **Thimbles,** in chimneys.
(34) **Throats,** in fireplaces.
(35) **Trap doors** and frames.
(36) **Trench covers** and frames, usually checkered plates (Art. 339).
(37) **Wheel guards.**
(38) **Wire grilles,** at windows, usually No. 10 gauge wire, $1\frac{1}{2}''$ diamond mesh, set in $1\frac{1}{2}''$ channel frames.

* Specifications, descriptions, and details of miscellaneous steel and iron work are given in Volume I.

334. Storefront Construction.

The essentials of storefronts are similar, although the materials for the finish may vary.

Since a store floor is usually only a few inches above the sidewalk grade, it first becomes necessary to provide light for the basement in the front of the store. This is solved by the use of a "bulkhead" or raised platform in the store window, which provides headroom for a basement sash and a show-front of convenient height. The entrance to the store is usually a pitched slab of concrete, sloping from the door to the building line to shed water. The shape of such entrances may be variously designed but the most usual is trapezoidal in plan. The desirability of a maximum glass area has prompted the invention of various compact sash

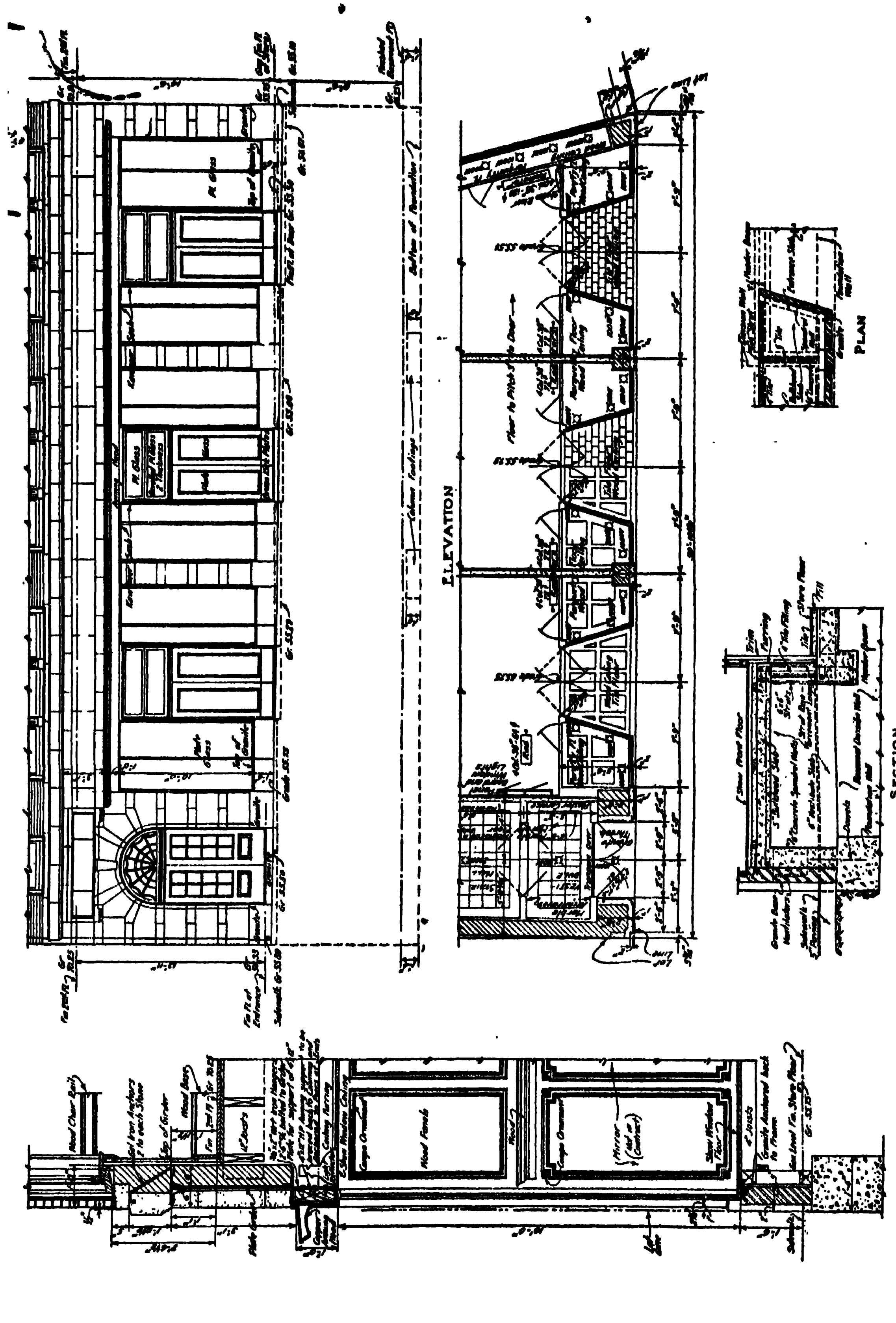


Fig. 528. Typical Storefront Construction

bars. This affects the column arrangement, especially in long storefronts, and necessitates stiff and watertight rebates for the plate glass.

A plan and a section through the bulkhead and spandrel girder of a typical storefront are shown in Fig. 528. A floor girder is usually carried across the store at about the line of the door. Upon this rests the entrance slab and the vertical inside wall of the bulkhead itself. The construction of the bulkhead is usually of 2 × 4's with metal lath and cement plaster on the exposed basement faces, as a fire protection, or concrete or terra cotta backing for the granite or marble finish. The store wall of the bulkhead, if plastered, should also have metal lath for a base. The floor of the bulkhead, or show-window platform, is also constructed of scantlings of required size, boarded over to receive the finish floor, and metal-lathed and plastered on the under side. This whole construction may, of course, be of concrete or tile and steel, and is advised when funds allow. As store fires usually originate in basements, a concrete bulkhead with hollow metal or metal covered basement sash is frequently an efficient check to their upward spread (Fig. 528).

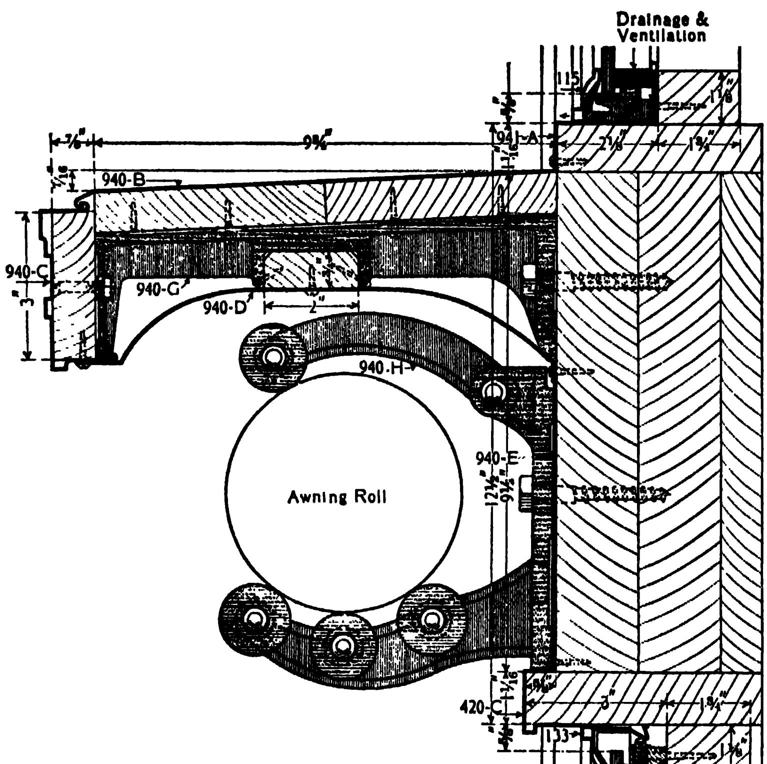


FIG. 529. AWNING TRANSOM BAR*

In Chap. 21, Book 1, are given some of the more usual plans of entrance arrangement. These are devised to provide as much show space as possible, the island type being often used to accentuate a feature. In the prevailing second-class store the entrance is framed in wood and the concrete base for the mosaic tile is cast upon this construction, as would be done with the ordinary bathroom tile floor. The better type of entrance is cast with reinforced slab bases for the tile which incidentally furnish a bearing for the cheek walls of the entrance. The door may be the ordinary hinged single or double door or it may be a revolving door, such as the Van Kannel.

The details of the various sash bars, rails, mullions, transom bars, and the like, are given in Chap. 21, Book 1. It must be remembered that these may be varied in many cases to suit other special needs, but they serve as a basis for such variation when desirable.

Spandrel girders are necessary over the storefront construction. These carry the walls above, so that a clear opening between the columns may be obtained below, in which the windows are placed. The weight of the latter construction is all carried at the sidewalk and bulkhead level.

It is usually necessary to provide a member between the columns to support the awning box, if an awning is used. Figure 529 shows one type of detail, which is an enlargement of a section similar to that shown in Fig. 528. In Fig. 529, the supporting member is hung from the spandrel girder. In other cases, the awning box may be somewhat lower, or it may be advisable and possible to erect its support after the main frame is considerably advanced, so that it could not be hung conveniently from the spandrel girder. In such instances, a steel supporting member (usually a channel) may be framed between the columns, independent of the spandrel girder, especially when concealed awning boxes are used. Such members should be made of ample size to carry the weight of the awning brackets, roll, and hood, the awning, and an assumed snow load in case the awning is down. The incident tendency to overturn the supporting member and an impact caused by sudden release of the awning should also be provided for. For usual cases, the member will be sufficiently strong if it is designed for a vertical load of 300# per linear foot.

335. Marquises.

In connection with hotel entrances, theatre lobbies, storefront construction, and so on, a marquise is often used. This consists of a framed

* Courtesy Zouri Drawn Metals Co.

canopy, either glazed* or opaque, which projects over the sidewalk and gives shelter at the entrance into the building. Figure 530 illustrates one type in which lookout beams cantilever from the building frame and form a support for the marquise. Figure 531 shows the other type, in which the outside portion of the marquise is supported by inclined rods or chains tied to a spandrel beam or to a column above, or by anchorage through the wall. The inside support of the marquise roof in the suspended type is a channel or angle anchored to the wall. The main structural support, however, is obtained by framing into a spandrel beam which is a part of the structural frame, or in some cases by framing directly into wall columns. The suspended type is more commonly used, as the main frame of the building is in reality a separate structure and where marquise work is made a later contract independent of the general contract for the structure, this type lends itself admirably.

The construction of a marquise is generally done by an ornamental iron company making a specialty of such work. In some instances, the structural steel framework is erected along with that of the building proper, and the marquise frame temporarily shored up. In any case, the structural engineer should be familiar with the general details of the work, as he may be called upon to coöperate with an architect in planning a marquise or to check shop drawings submitted for approval.

The construction of a marquise usually consists of a frame of structural steel channels around the edges, and if the projection from the face of the building is too great, an intermediate channel is framed between the two side channels. Structural steel tees (commonly $2\frac{1}{4}'' \times 2\frac{1}{4}''$ or $3'' \times 3''$, depending upon the span) are framed between the channels, parallel to the edges of the marquise and usually 16″ o.c. All joints should be rigidly bolted and the corner connections of the channels should be made with gusset plates to give added rigidity. The flanges of the tees may be covered with copper, either plain or paneled, as shown in Fig. 532. Between the tees, $\frac{1}{4}''$ rough-surface, wired, plate glass is bedded in putty specially prepared for the purpose, as shown.* Where the edges of the glass occur, small metal strips are inserted to make a tight joint. These are also puttied. The projecting stems of the tees are covered with 16 oz. copper capping, as shown in Fig. 532, held in place by bolts through the tees. The outside channels (or for short spans, angles may be used) are covered by a bronze or cast-iron facia, moulded, paneled, or with glazed serrations according to the detail desired. The whole marquise is inclined toward the building with a pitch of about $\frac{1}{4}''$

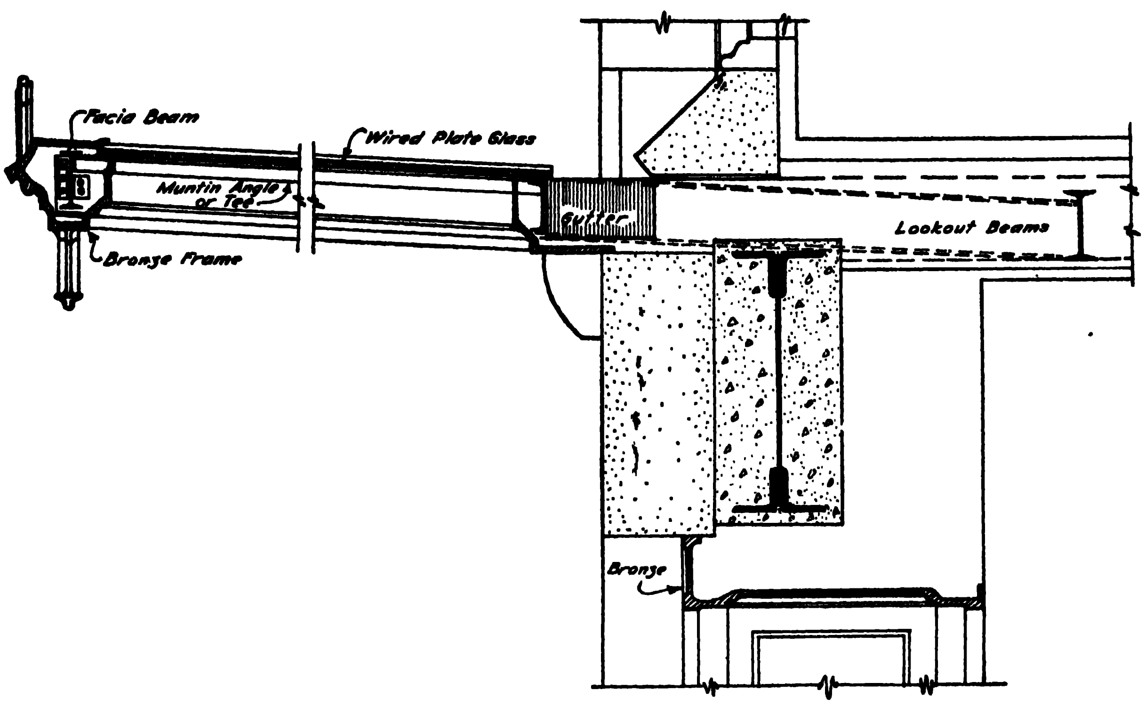


Fig. 530. Marquise

* Glazing is now generally considered less satisfactory, and the tendency in better types of marquises is to use 3″ cinder concrete slabs between the tees covered by a roofing material, with a paneled soffit of colored cement plaster and bronze lighting fixtures.

per foot. In the rear, at the building line, a gutter is placed with goosenecks leading to conductors inside the building. A ¾″ mesh copper screen is sometimes placed above the tees to protect the glass from falling objects. The ties in the suspended type may be either chains or rods. Rods with turnbuckles for adjustment are better, as less vibration is probable. The rods may be twisted or ornamented as desired for effect, but a minimum section of 1″

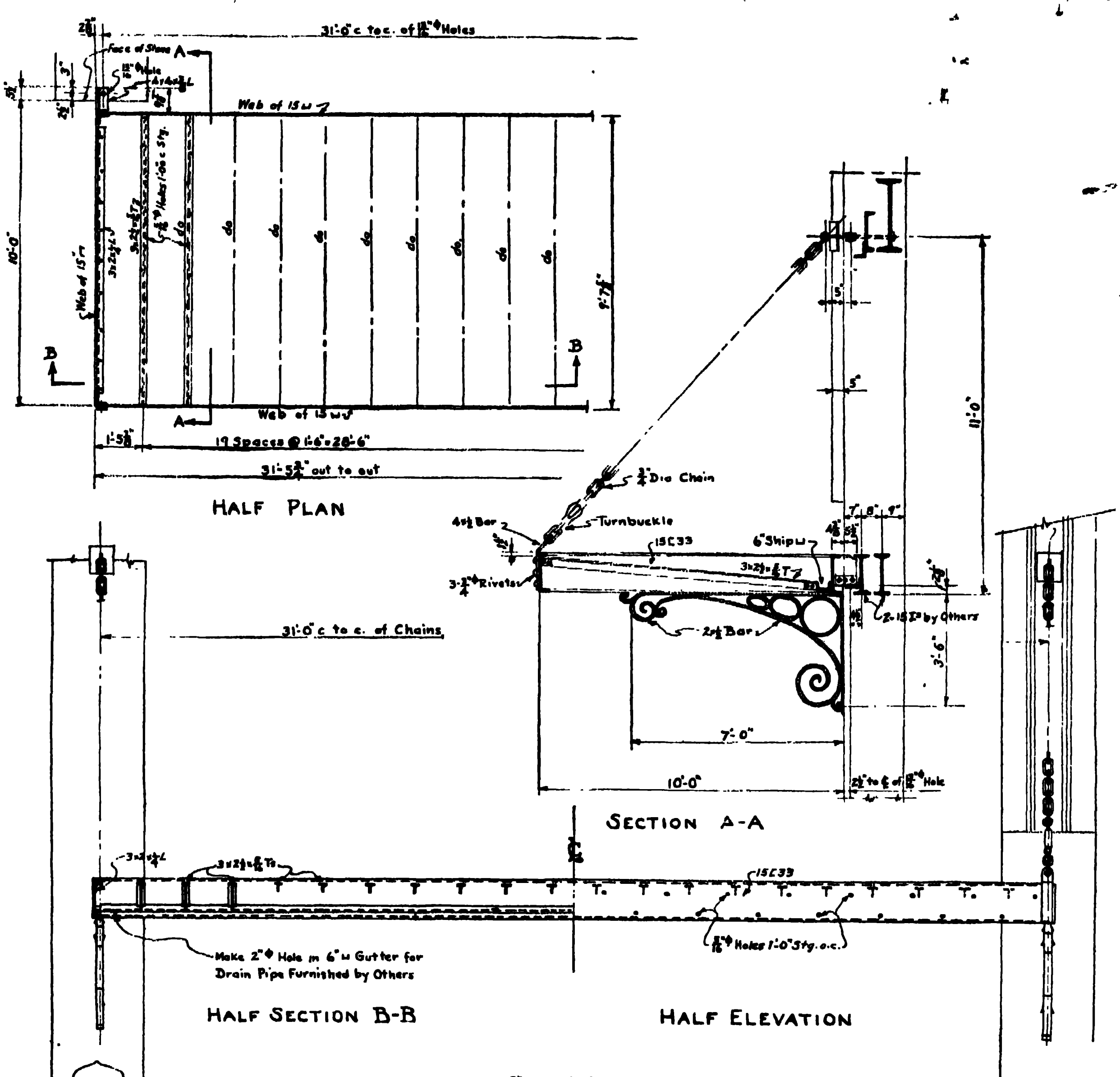


Fig. 531*

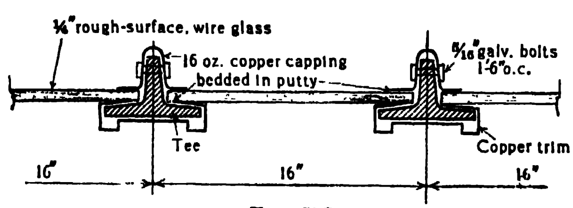


Fig. 532

should be maintained for practical reasons. Rings with strap attachments may be connected to the channels at the bottom and to the anchorage at the top, and the rods installed later, if preferred.

Illustrative Prob. 335a. Check the typical sizes of members shown in Fig. 531.

L.L. = 40	Use T's 18″ o.c.
Glass and Supports = 10	Ld./ft. on T = 50 × 1.5 = 75#
T.L. = 50#/□′	Span of T's = 9′-7⅝″ = 9.63′

* Courtesy of F. A. Norcross, Architect, Boston, Mass.

$M = 1.5\,w \cdot L^2 = 1.5 \times 75 \times (9.63)^2 = 10{,}460''\#$

$\frac{I}{c} = \frac{10{,}460}{16{,}000} = 0.66''^3$

Use $2\frac{1}{2} \times 3 \times \frac{5}{16}$ T's $\left(\frac{I}{c} = 0.72''^3\right)$

$2\frac{1}{2}''$ flange, $3''$ stem

$D = \frac{5\,W \cdot l^3}{384\,E \cdot I} = \frac{5 \times (75 \times 9.63) \times (9.63)^3 \times (12)^3}{384 \times 30{,}000{,}000 \times 1.5} = 0.32''$ actual

$D = \frac{l}{360} = \frac{9.63 \times 12}{360} = 0.33''$ allowable

Deflection O.K.

Front and Rear Channels

Ld./ft. $= \frac{50 \times 9.63}{2} = 241\#$ Span $= 31'\text{-}5'' \pm$

$$\begin{aligned} \text{Channel} &= 34 \\ \text{Facia} &= \underline{20} \\ \text{T.L.} &= 295\#/\text{ft.} \end{aligned}$$

$M = 1.5\,w \cdot L^2 = 1.5 \times 295 \times (31.4)^2 = 437{,}000''\#$

$\frac{I}{c} = \frac{437{,}000}{16{,}000} = 27.4''^3$

Use 15 [33.9 $\left(\frac{I}{c} = 41.7''^3\right)$

$L = 2\,d = 2 \times 15 = 30'$ allowable

$L = 31.4'$ actual

Deflection O.K.

(15'' [not fully stressed)

End reaction $= \frac{295 \times 31.4}{2} = 4630\#$

$$\begin{aligned} \text{Side channel, from roof} = 50 \times \tfrac{9}{12} &= 37 \\ \text{Channel} &= 34 \\ \text{Angle} &= 4 \\ \text{Facia} &= \underline{20} \\ \text{T.L.} &= 95\#/\text{ft.} \end{aligned}$$

$\frac{95 \times 9.63}{2} = 460\#$. Total $= 460 + 4630 = 5090\#$.

Length of chains — c.c. of attachments

$(9.42)^2 = 89$ $\quad \sqrt{184} = 13.6'$

$(9.75)^2 = \underline{95}$

184

Tension in chain $= T$

$$\frac{13.6}{9.75} = \frac{T}{5090} \text{ or } T = 7100\#$$

Allowable tensile stress for wrought-iron $= 12{,}000\#/\square''$.

Area required $= \frac{7100}{12{,}000} = 0.59\square''$

Use links of $\frac{3}{4}''\phi$ stock

$(2 \times 0.44 = 0.88\square'')$

Use Std. turnbuckles

Connection plate. Assume 4'' width and $\frac{7}{8}''$ rivets.

$4'' - \frac{7}{8}'' = 3\frac{1}{8}''$ net width. Area required $= \frac{7100}{16{,}000} = 0.45\square''$.

Thickness required $= \frac{0.45}{3.12} = 0.145''$.

Use $4 \times \frac{1}{2}$ Pls. for practical reasons.

Conservative areas of metal should be provided on account of the positive chances for rusting.

Use 3-$\frac{3}{4}''\phi$ rivets.

Prob. 335b. Determine the sizes required for a marquise similar to the arrangement shown in Fig. 531 if the projection is to be 8'-0'' and it is to be 20'-0'' long (parallel to the building). Use T's 16'' o.c. supporting a 2'' concrete slab and a tar and gravel roofing, with plaster and metal lath applied under the ties. Use single twisted square rod suspenders (allow for area at root of threads to turnbuckles). L.L. $= 40\#/\square'$.

336. Flagpoles.*

The design of flagpoles in present practice has become the province of special manufacturers, but in special cases, when large poles are planned for buildings, the structural engineer should understand the important features and be able to check them up if necessary, and to make suitable arrangements in the framing for supports and braces.

Wood poles are no longer commonly used, as spar lengths of timber are scarce in most localities, and transportation by rail from points where such material is available is costly. If floated by rafts,

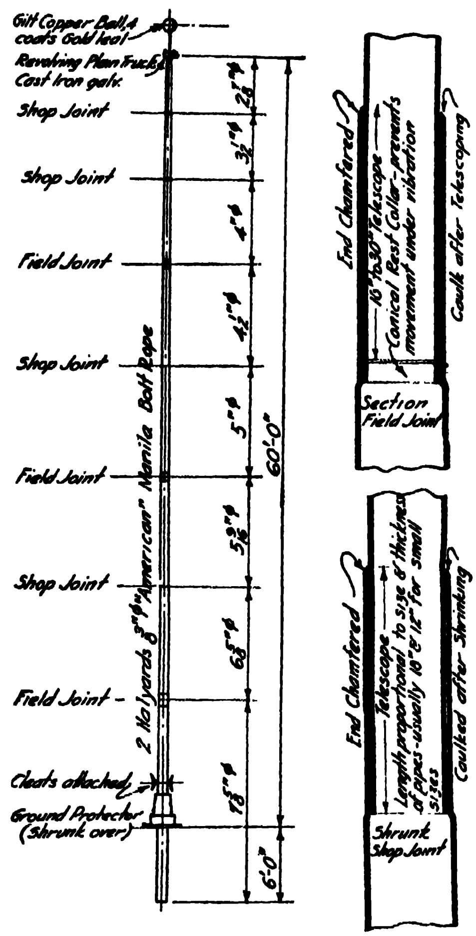


FIG. 533

the wood becomes water-soaked and requires careful seasoning afterwards, or checks will result, which may lead to leaks around roof poles, or to rotting at the bases of ground poles. A wood pole splinters badly if struck by lightning and thus becomes a source of public danger. Poles of structural steel (such as latticed angles) are expensive, present a poor appear-

* Some of the notes in this article and all of the figures are based (with permission) upon data given in a pamphlet issued by the Pole and Tube Works, Inc., Newark, N. J.

ance, and are inaccessible for field paint renewals near the top. The ideal would be a continuous steel tube of conical shape. This cannot be obtained by ordinary manufacture. It is generally agreed that the best approach to the ideal is a series of steel tubular pipe sections of varying diameters, well joined together, as illustrated in Fig. 533.

The actual lengths of the sections have little effect upon the strength, although the butt section should not be too short. The important feature is to have sufficient section at each point to resist the stresses safely. Figure 533* illustrates typical shop and field joints. The following* describes their fabrication:

Shop Joints. The larger pipe is cut to length and chamfered to a 30° angle with a sharp edge. It is then heated and pushed through a bell die, thus "swaging" (reducing) its diameter to equal the outside diameter of the next lower section. While it is hot, it is pushed with a 25 ton pressure 18″ over the next section and allowed to shrink on. The chamfer is then caulked tight.

Field Joints. The larger pipe is cut and chamfered in the same manner as for the shop joint and swaged over a cast-iron mandrel 24″ long. Reheating the pipe loosens it from the mandrel. The pipe when hot then has a conical rest collar pressed into position. The lower section is then match finished to the collar. During erection, the joint is caulked.

Undesirable Joints and Their Disadvantages. Figure 534 shows other methods of making joints. In (*a*), the threads reduce the bending resistance about 40%, so that excessively thick pipe would be required. In (*b*), the coupling must be quite heavy to equal the strength of the pipe. The result is clumsy, expensive, and difficult to assemble. The joint in (*c*) has merit, but is expensive, the same as in (*d*), — the latter giving an odd appearance to the pole. In (*e*), the shrinkage must be made in a vertical position, or else considerable pressure for the lead must be used. Strains tend to compress the lead and water may be gradually admitted, with incipient rusting. In (*f*), the strength of the pins is weak compared with the pipe. Any movement of the pins induces the paint film to crack and rusting follows. The top lead caulking may also be inefficient.

The design of a pole involves several factors. First, the height of the pole should be proportional to the height of the building, if a good appearance is to be had.† The size of the flag should be proportional to the height of the pole, and some authorities make the horizontal width of the flag equal to $\frac{1}{8}$ the height of the pole. Flag sizes are quite well standardized by feet as follows:

2 × 3, 2½ × 4, 3 × 5, 4 × 6, 4 × 7, 5 × 8, [illegible] × 9½, 6 × 9, 6 × 10, 6 × 12, 8 × 12, 9 × 15, 10 × 1[illegible], 10 × 19, 12 × 18, 12 × 20, 15 × 25, 20 × 30, and 2[illegible] × 38.

The wind pressure must be calculated. From tests,‡ the horizontal pull of a flag is empirically established as:

$r = 0.0003\, v^2$, in which
v = the velocity of the wind in miles per hour, and
r = the pull per □′ of flag surface.

For a maximum wind velocity of 100 miles per hour, which is an average of the extreme amounts recorded (Art. 160), $r = 3\#/□'$.§ Having the area of the flag and the unit pull, the force may be calculated, and assumed as applied at the center of the vertical height of the flag.

The wind pressure upon the pole itself must be qualified in certain ways.¶ Referring to a common formula for the wind pressure on a flat surface perpendicular to the direction of the wind, which is

$p = 0.004\, v^2$, in which
p = the pressure in #/□′, and
v = the velocity of the wind in miles per hour,

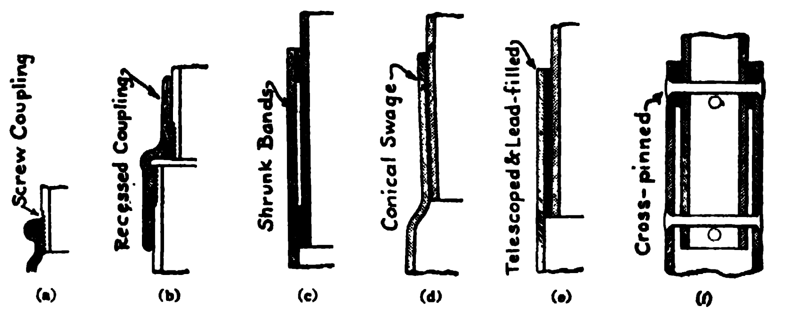


FIG. 534

the pressure for a 100-mile velocity is

$$p = 0.004(100)^2 = 40\#/□'.$$

On a cylinder, the wind slides off of the surface, and the unit pressure is only about ½ that on a flat surface.‖ Referring to the above, a conservative unit pressure of 25#/□′ on the vertical projection of the pole is recommended. The diameter of the pole will vary, and on the basis of an average diameter

* Based upon recommendations of the Pole and Tube Works, Inc., Newark, N. J. This type of joint is claimed to be as strong as the parts joined, and "drop" tests, pulling or pushing tests, and bending tests have been made to verify the results.

† This will depend upon the proportion of the building but a ratio of pole to building height is often made 3 to 7.

‡ Tests in 1916 by Bureau of Yards and Docks, U. S Navy.

§ Also recommended by Mr. R. Fleming.

¶ Precautions should be taken to avoid having the flag wrap around the pole or foul the halyards, as the flag becomes torn. Patented halyard tops which adjust themselves to the wind are helpful.

‖ Theoretically, the coefficient of v^2 in a formula for wind pressure should be constant for geometrically similar bodies, presented in the same manner to the wind, regardless of size, but the coefficients do not remain constant according to tests. Above 3″ diameters, the variation is less and according to tests described in Scientific Paper #394, U. S. Bureau of Standards, Washington, D.C., by H. L. Dryden, the coefficient is about 0.43.

of 7½″ for all lengths, a pressure of 15#/ft. of height is recommended. Any pole should be strong enough to resist not only the commonly occurring but also the exceptionally violent local storms. The bending moment at any point in the height may then be calculated for a cantilever with a uniform load over its length and a concentrated load at the free end. The resisting moment is

$$M_r = \frac{s \cdot I}{c} = \frac{s \cdot \pi(D^4 - d^4)}{64\,D}\text{, in which}$$

D = the outside diameter in ins., and
d = the inside diameter in ins.

Table 84 gives properties for hollow pipe which are useful in this work. The difference between the

TABLE 84

DATA FOR STEEL TUBULAR PIPE

Nominal Size (Ins.)	Thickness (Ins.)	Weight per Foot (#)	I (Ins.⁴)
2½	0.203	5.8	1.53
3	.216	7.6	3.02
4	.237	10.6	7.23
5	.258	12.8	15.16
6	.280	19.0	28.14
7	.301	23.5	46.51
8	.322	28.6	72.49
9	.342	33.9	107.6
10	.365	40.5	160.7
11	.375	45.6	217.0
12	.375	49.6	279.3
13	.375	54.6	372.8
Extra Strong			
2½	0.276	7.7	1.92
3	.300	10.3	3.89
4	.337	15.0	9.61
5	.375	20.8	20.67
6	.432	28.6	40.49
7	.500	38.0	71.37
8	.500	43.4	105.7
9	.500	48.7	149.6
10	.500	54.7	211.9
11	.500	60.1	280.1
12	.500	65.4	361.5
13	.500	72.1	483.8

outside diameter of the inserted pipe and the inside diameter of the larger pipe should be kept a reasonable amount in order to avoid heavy swaging. The same strength may be obtained by the use of thinner pipe of a larger diameter, although minimum thicknesses should be fixed on account of probable corrosion. The diameters should be so selected that a good appearance will result. Table 85* may serve as a guide to diameters, and to jointing. Lengths of 20′-0″ for the pipe are readily available, and shipping lengths of 40′-0″ are common, but it is usually more economical to make the joints closer than this and to connect every other joint in the field.

TABLE 85

GENERAL PROPORTIONS FOR POLES

Height (Ft.)	Diameter (Ins.)	Setting Length (For Ground Poles) (Ft.)
20	4	3
25	4½	3½
30	5	3½
40	5½	4
50	6½	5
60	7½	6
70	8½	7
80	11	8
90	12	9
100	13	10

The pole must be properly seated and connected with the structural frame. The member supporting it should be designed to carry the weight (refer to

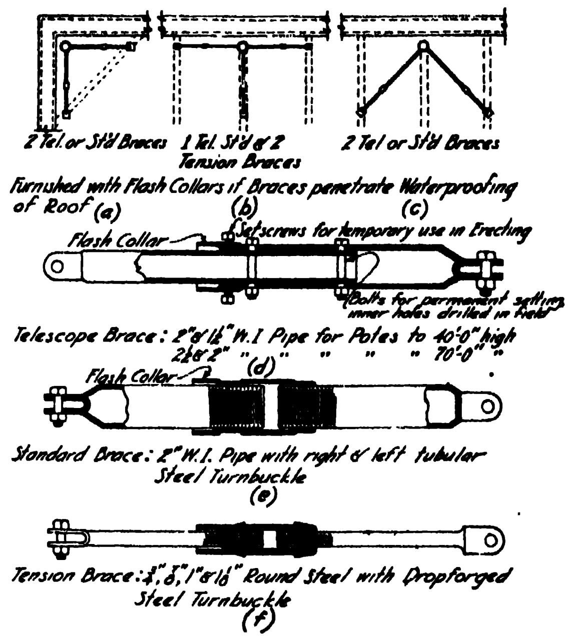
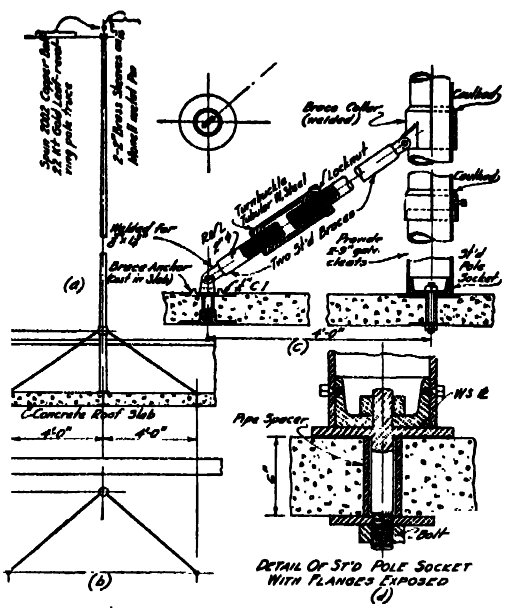


FIG. 535

Table 84 for unit weights). In addition, good anchorage should be provided for the braces. Figure 535 (*a*), (*b*), and (*c*) shows typical arrangements of

* The pipe should be proportioned to have its weakest (yet safe) point at about the middle of the pole length. Thus, when a pole is accidentally overloaded, it will bend near the middle rather than falling the full length. The base section should be excessively strong. The strength of the pole as a column is generally very ample.

braces, while (*d*), (*e*), and (*f*) illustrate different types of braces. Figure 536 (*a*), (*b*), and (*c*) shows details for an installation where the pole socket is cast into the slab. For a socket placed later, (*d*) shows the detail. Figure 536 (*e*) and (*f*) shows connections for wood-framed roofs, and (*g*) an adaptation to steel roof beams. Some engineers prefer to have the pole carried down through the roof, as illustrated in Fig. 537, and omit the braces. This may be done



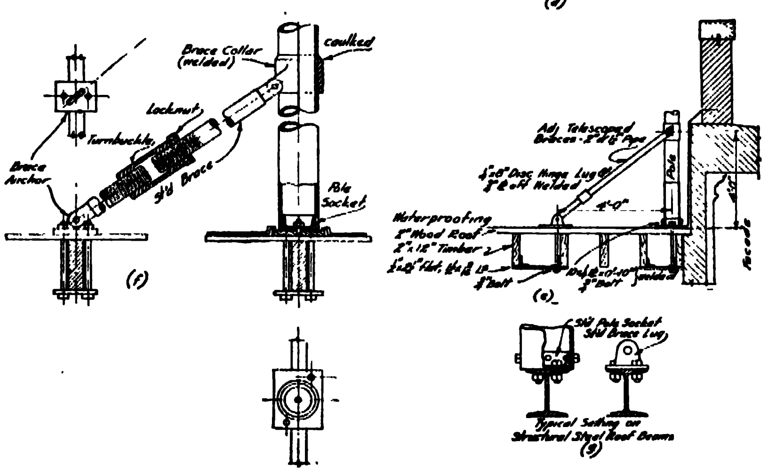


Fig. 536

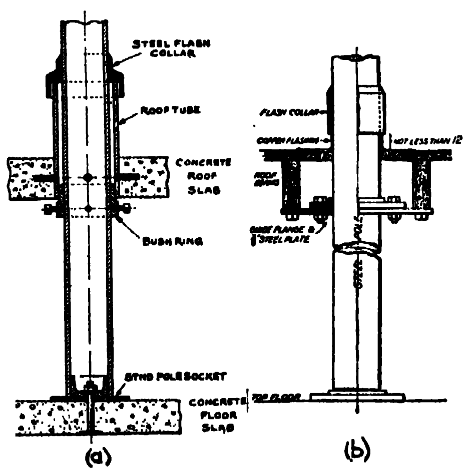


Fig. 537

if the pole does not interfere with the space. If a suspended plastered ceiling exists under an attic space, it is better to support the pole on a roof beam to avoid plaster cracks.

Illustrative Prob. 336a. Design a steel tubular pole for a 50'-0" length.

Width of flag $= \frac{1}{10} \times 50 = 5.0'$. Assume $5' \times 9\frac{1}{2}'$ flag.
Area $= 5 \times 9.5 = 47.5\square'$.

Pull on flag $= 3\#/\square' \times 47.5 = 142.5$, say $150\#$.
Load per ft., wind pressure $= 15\#$.
For wind pressure,

$$M = \frac{w \cdot L^2}{2} = \frac{15 \times (50)^2}{2} = 18{,}750'\#.$$

For flag pull,

$$M = P \cdot L = 150 \times 47.5 = \underline{7{,}125}$$
$$\text{Total } M = 25{,}875'\# = 310{,}500''\#$$

$$\frac{I}{c} = \frac{310{,}500}{16{,}000} = 19.9''^3. \text{ Requires } 6''\ \phi \text{ section.}$$

Refer to Table 85 for diameters at other points of height.

Horizontal reaction at base $= 150 + 15(50) = 900\#$.
Use type of bracing in Fig. 535 (*c*) and (*e*).

Force in brace $= \frac{900}{2} \div 0.707 = 640\#$.

Approximate weight of pole:

Top–20'	$= 20'$ @ $4''\phi$	$= 20 \times 10.6\#/\text{ft.}$	$= 212$
20–25'	$= 5'$ @ $4\frac{1}{2}$	$= 5 \times 11.5$	$= 58$
25–30'	$= 5'$ @ 5	$= 5 \times 12.8$	$= 64$
30–40'	$= 10'$ @ $5\frac{1}{2}$	$= 10 \times 19.0$	$= 190$
40–50'	$= 10'$ @ 6	$= 10 \times 23.5$	$= 235$
			759, say 800#.

Design supporting beam for a vertical load of 800# and anchoring beam for two inclined forces of 640# each.

Ground poles require proper footings. Table 85 gives minimum lengths of embedment, which should approximate 10% of the pole height above the ground. The depth of the footing should be influenced by soil conditions and frost lines. Figure 538 gives recommendations for a typical case. Stock pattern, or specially designed, pedestals of bronze, gray or galvanized iron may also be used. The ring moulding at the top of the base should be cast separately so that the pedestal may be filled with grout to prevent rusting of the pole inside. Without pedestals, ground protectors (dog guards)

should be used. These consist of a larger and thicker piece of pipe 2'-0" long, slid over the butt end of the pole and swaged on to extend 16" above the ground surface. Window poles are sometimes

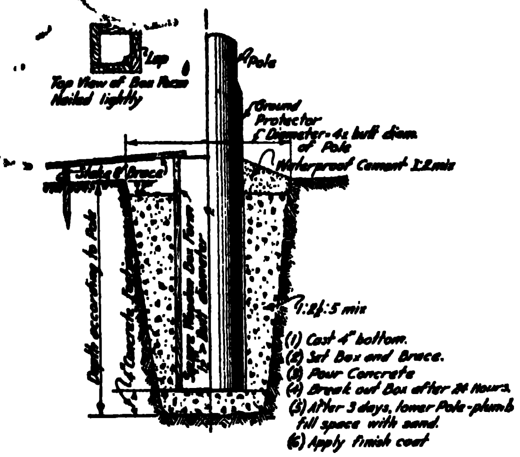


Fig. 538

used, as shown in Fig. 539. These may be equipped with stock fittings, and should be well anchored to the wall, unless insignificant in weight.

Prob. 336b. Design a roof pole to be 60'-0" high, and make typical sketches if it is to be installed on a concrete framed roof.

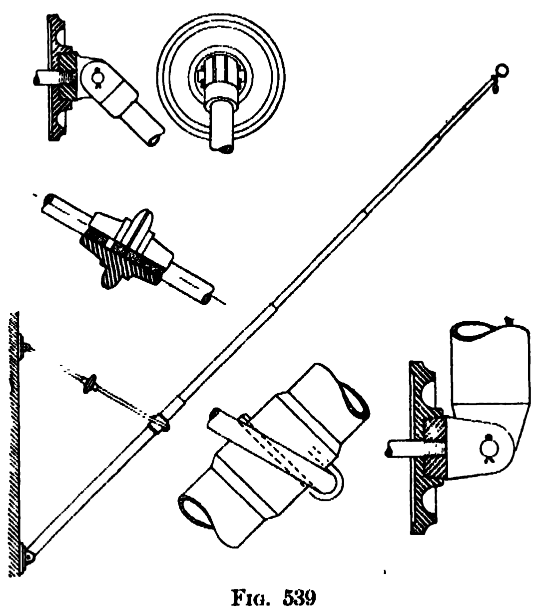


Fig. 539

337. Sidewalk Vault Lights and Floor Lights.

SPECIFICATION CLAUSES*

Vaults Under Sidewalks Where a vault is built under the sidewalk a wall shall be constructed to retain the adjacent banks.

The roofs of all vaults shall be of approved incombustible material. Glass, when used in the roofs of the vaults, shall measure not more than 16 square inches in one light.

All vaults shall be thoroughly ventilated.

Sidewalk Loads For sidewalks between the curb and building lines, live loads shall be taken at 300 pounds per square foot or a concentrated load of 5 tons at any point.

Floor Lights Floor lights shall have metal or reinforced concrete frames, and shall be of the same strength as the floors in which they are placed. The glass in floor lights shall be not less than ¾ inch in thickness, and if any glass measures more than 16 square inches, there shall be a wire mesh, either in the glass or under it.

City buildings are often required to be set back from the curb lines and large areas of basement floor may be made more useful by introducing prismatic glass in the sidewalk to refract daylight and thus partially eliminate the use of artificial light. Figure 540 gives typical details. In general, the panels consist of a body of concrete reinforced with steel bars set between rows of glass and around the outer edges. The glass in the 3-way system is of 4" square tiles with a series of prisms on their faces. The glass in other types is usually of 3¼" square plain or multiprism lens, 3¼"ϕ plain lens, 2¾" square plain or multiprism lens, or 4" square target lens.

In planning the lights, the rectangular areas which are to be used for lighting should be indicated on the ground floor plan and the framing to support the panels clearly shown. Steel or concrete beams are commonly used for this purpose, one parallel to the building at the wall with cross beams at the butting edges of the panels, as shown in Fig. 540. The cross beams and the outside edge of the sidewalk lights are supported by the retaining wall terminating the basement. The layout is often worked out in conjunction with the advice of the company that is to furnish the type of vault light specified. The area provided with lights should not be less than 20% of the sidewalk area affected.

The local building code should always be consulted relative to the live load requirements in the design of the framing of this work. Code requirements usually make it necessary to investigate the sidewalk slab or its incident frame for both a uniform load of from 250# to 300# per square foot and a concentrated load of 8000# or more, considered separately. From these calculations the member is designed for the controlling condition of both moment and shear. The dead load of sidewalk light construction averages 30#/□', exclusive of an allowance for the weight of the supporting members. The design of the beams is straightforward enough, if their location in plan and elevation is definitely known to conform with the details.

* Building Code of the National Board of Fire Underwriters, New York City.

Cast-iron sidewalk lights are also occasionally used which are constructed of cast-iron panels glazed with various sizes of glass set in concrete, or in panels with projecting iron knobs and glass set with elastic cement. These may be furnished with panels only or with panels and cast-iron outer framework.*

In certain installations, sidewalk doors are used in connection with this work. These are manufactured in flush or raised types, with either checkered steel or plain steel surfaces. They may also be made with various sizes of glass set in cast-iron construction. These doors are fitted with brass hinges, flush lift-rings, devices to hold them in place while open, and a slide-bolt lock underneath. Flush door frames are cast with a gutter to carry off water which may seep through the joints of the doors. Some installations include automatic self-opening and self-closing doors with a spring guard gate and a telescopic hoist operated by a motor.† In all cases, beams are usually framed around the door openings to provide the necessary supports.

Prismatic lights may also be used in connection with interior floor construction in special cases, if desired. This is quite common in the main floors of libraries and in other public buildings.

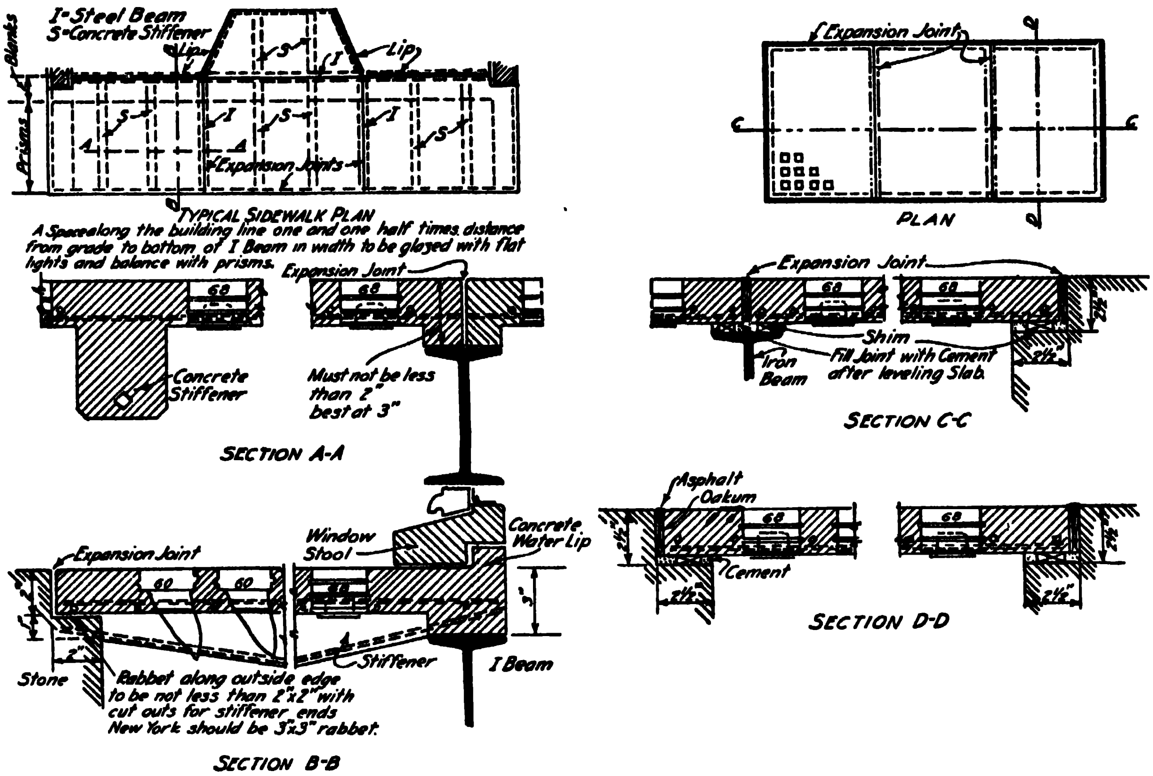


FIG. 540. SIDEWALK VAULT LIGHTS §

338. Gratings.

SPECIFICATION CLAUSES‡

Areaways, Projections Beyond the Building Line 1. Areaways or openings covered with iron gratings or with iron doors not more than 3 feet in width, with rough surface set flush with sidewalk, may project not more than 4 feet beyond the building line. If gratings be used, they shall have a wire screen of not more than ½-inch mesh securely attached to the under side.

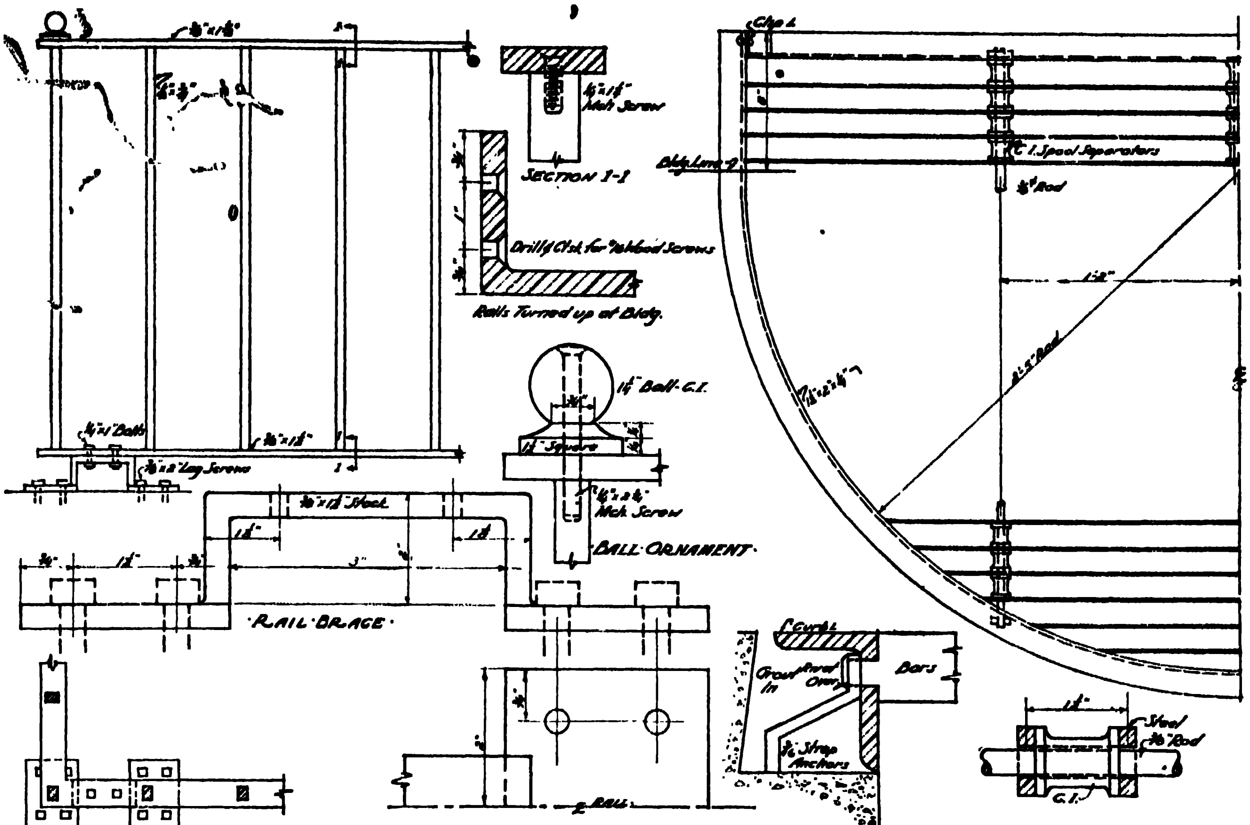
2. No open areaways, railings, steps, or any portion of a building or structure shall project beyond the building line at any point less than 10 feet above the curb level.*

Gratings are used for covering areaways, for balconies, grids and pit floors in boiler rooms, and so on. These are generally made up of small steel bars, usually 1" deep and of the required thickness, extending in the transverse direction, through which $\frac{3}{8}''\phi$ rods run at a spacing of about 1'-6". The bars are kept the proper distance apart, commonly $1\frac{3}{16}''$ c.c., by cast-iron spool or pipe separators, as shown in Fig. 541. Other patented types¶ have small cross-bars instead of the separators, which are fastened by lock joints made by pressure. The gratings are supported by curb angles, as shown in the figure. Table 86 is useful in determining safe loads.

339. Checkered Plates.

Checkered plates, such as illustrated in Fig. 542, are sometimes used for treads and landings of service stairs, covers for pipe trenches and pits, floors of

* Richards and Kelley Mfg. Co., Chicago, Ill.
† Gillis and Geoghegan Hoists.
‡ The Building Code of the National Board of Fire Underwriters, New York City.
§ Courtesy American 3-Way System.
¶ There are several types, such as Mitco, Tri-Lok, Reticuline, Irving Subway, etc.



RAILING AREA GRATING

FIG. 541. DETAILS OF RAILS AND WROUGHT IRON GRATING

TABLE 86*

SAFE LOADS FOR "MITCO" GRATINGS

Safe loads in pounds per square foot based on fiber stress of 16,000 pounds per square inch

Depth (in.)	Bearing bars		Cross bars		Weight per sq. ft. (lb.)	Span in feet										
	Size (in.)	Spacing c. to c. (in.)	Size (in.)	Spacing c. to c. (in.)		2	2½	3	3½	4	5	6	7	8	9	10
1	1 × 3/16	1 3/16	7/8 × 1/8	4¾	7.8	835	534	370	273	208	133	93				
1¼	1¼ × 3/16	1 3/16	1 × 1/8	4¾	9.5	1410	900	627	445	352	225	157	111			
1½	1½ × 3/16	1 3/16	1 × 1/8	4¾	11.0	1875	1200	834	612	468	300	208	153	117	93	
1¾	1¾ × 3/16	1 3/16	1 × 1/8	4¾	12.5	2560	1640	1140	838	640	410	285	210	160	127	103
2	2 × 3/16	1 3/16	1 × 1/8	4¾	14	3340	2140	1480	1090	834	534	371	272	208	165	133
2¼	2¼ × 3/16	1 3/16	1 × 1/8	4¾	15.5	4220	2700	1875	1380	1055	675	470	345	264	208	169
"MITCO" DRIVEWAY GRATINGS (FOR HEAVY TRAFFIC, COAL "GRIZZLIES," ETC.)																
2¼	2¼ × 3/8	1 3/16	2 × 5/16	4¾	36.8	8440	5400	3750	2760	2100	1350	940	690	528	416	338
2¾	2¾ × 3/8	1 3/16	2¼ × 5/16	4¾	44.6	13900	8880	5640	4530	3470	2220	1540	1130	865	685	555
3¼	3¼ × 3/8	1 3/16	2¼ × 5/16	4¾	51.7		12400	7850	6310	4840	3100	2150	1580	1210	955	775
3¾	3¾ × 3/8	1 3/16	2¼ × 5/16	4¾	58.7		16500	10470	8410	6440	4120	2862	2100	1610	1272	1031
"MITCO" ARMORGRIDS (FOR ARMORING CONCRETE PLATFORMS AND FLOORS)																
1	1 × 3/16	2⅜	7/8 × 1/8	2⅜	5.5	416	266	186	136	104						

* Based upon a similar table prepared by the Hendrick Mfg. Co., Carbondale, Pa. "Mitco" gratings are a patented type and are made of bars in both directions connected together. The grating shown in Fig. 541 is not a "Mitco," but is simply a forge shop type.

balconies in boiler rooms, and so on. In planning such details, the engineer is often interested in providing sufficient thickness for a given span and load, or a certain spacing of supports for a given thickness.

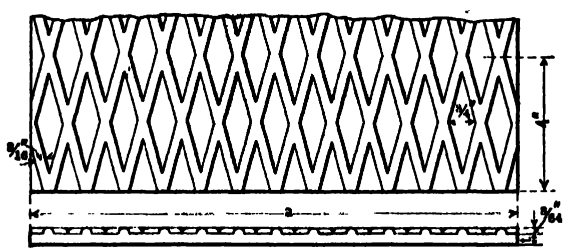


FIG. 542

Table 87 gives the common sizes and the net safe loads for various spans, when the plates are supported on two sides only. Widths up to 12″ are available in 10′-0″ maximum lengths and larger widths are available in 20′-0″ maximum lengths. The load values may be checked by the usual procedure.

TABLE 87

SAFE LOADS FOR CHECKERED PLATES #/□′

s = 16,000#/□″ supported on 2 sides only

Span in Feet	Thickness (″)	$\frac{1}{2}$	$\frac{7}{16}$	$\frac{3}{8}$	$\frac{5}{16}$	$\frac{1}{4}$	$\frac{3}{16}$
	Wt. — #/□′	21.4	18.9	16.3	13.8	11.2	8.7
	$\frac{I}{c}$ — (ins.)³	0.50	0.38	0.28	0.19	0.13	0.07
1	Minimum widths 12″ Maximum widths 60″ (except for $\frac{3}{16}$″ Pl. which is 48″.)	5310	4060	2985	2065	1320	740
2		1310	1000	735	505	320	180
3		570	435	315	215	140	75
4		310	235	175	115	70	40
5		190	140	105	65	40	
6		130	90	65	45		
7		90	60	45			

Illustrative Prob. 339a. Check the safe load per □′ for a $\frac{3}{8}$″ plate on a 5′-0″ span.

Test a strip 12″ wide. $\frac{I}{c} = \frac{b \cdot d^2}{6} = \frac{12 \times (0.37)^2}{6} = 0.28''^3$

$$M = 1.5\,w \cdot L^2 = s \cdot \frac{I}{c} = 1.5\,w(5)^2 = 16{,}000 \times 0.28.$$

$w = 121\#/□'$ total.

Weight of plate = 16

105 #/□′ = net safe load.

When plates are supported on four sides, the design should theoretically be made by the method of flat plates,* but this is rather involved for practical work, and the formula which is used in two-way slab design may be used, namely,

$$r = \frac{l}{b} - 0.5, \text{ in which} \qquad (S\text{-}87)$$

r = the ratio of the load carried in the transverse direction,

l = the span in the longitudinal direction, and

b = the span in the transverse direction.

* See Fuller and Johnson's "Applied Mechanics" — John Wiley and Sons, Inc., Publishers.

340. Library Stacks.

In libraries, universities, court houses, and so on, provision must be made for the storage of many books. In modern buildings, this is done by the use of standard steel bookstacks, as shown in Fig. 543. These consist of sheet-steel shelves supported by winged brackets, which are carried by 2″□ central uprights of steel tubing. For heights exceeding 3 tiers, the latter are reinforced with heavy H sections inside of the tubing. The shelf spaces are made 8″, 10″, or 12″ deep, 8″ being common. Double-faced shelf spaces are 16″ wide (for 8″ shelves) and wall-shelf spaces are 10″ deep. The uprights are spaced 3′-0″ o.c., and a tier is 7′-6″ high overall, or 7′-6″ floor to floor, as a common standard. One such grouping of shelves is called a section, and a number of sections in a row (usually 4) is called a range. The ranges are spaced 4′-6″ o.c. in a lateral direction, although 4′-0″ o.c. may be used when the stacks are only accessible by the library staff. This gives about a 3′-1″ clear range aisle (Fig. 544). The main stack aisle should preferably be 3′-6″ wide and not less than 3′-0″ wide if book trucks are to be used. The end aisles should not be less than 2′-6″ wide, and 3′-0″ is better if available. When wall ranges are used, the distance from the face of the wall to the center of the first interior range may be established with the above data.

With the surrounding walls of the room or rooms located, and the available height fixed, the architect makes a stack room layout, similar to Fig. 545. The available space, fixtures, and conveniences are studied to make them correspond with the number of books which are to be housed, viewing present and future needs. The number of volumes which may be accommodated may be approximated from the following tabulation:

Type of Book	Volumes per Ft. of Shelf	Volumes per 3 Ft. Section (double-faced)
Fiction	9	378
General Literature	8	336
Reference Books	7	294
Law Books	5	210

FIG. 543*

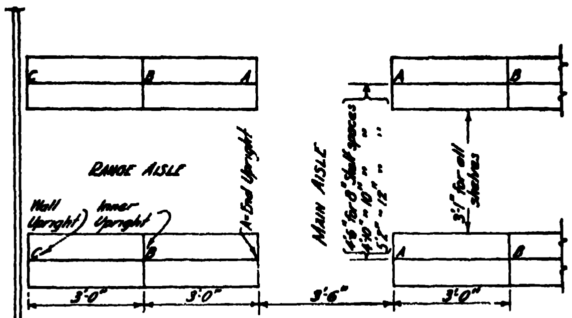


FIG. 544*

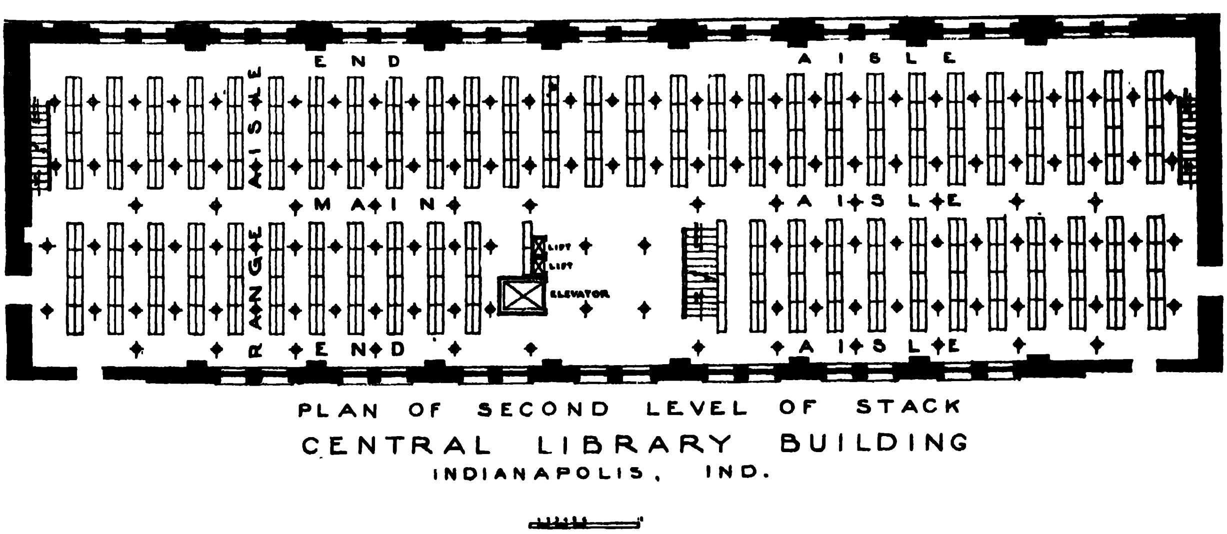


FIG. 545*

* Courtesy of the Library Bureau, Boston, Mass.

In the usual case, stack rooms are planned to contain more than one tier in height. The inter-

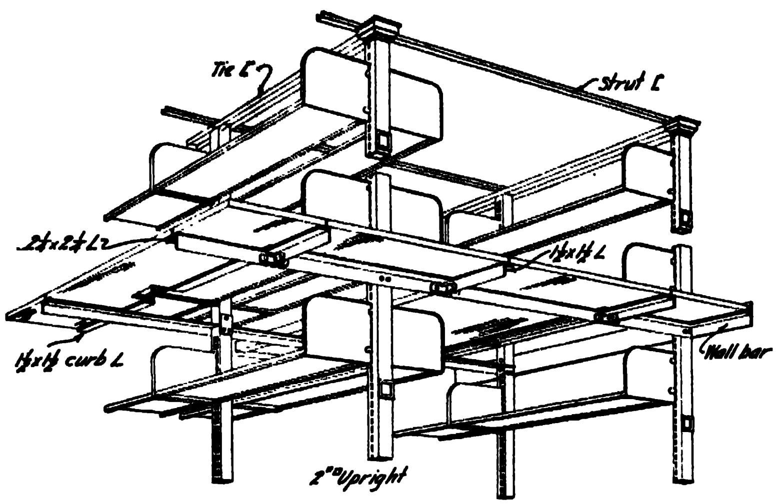


Fig. 546*

mediate floors (7'-6" apart) are carried by the stack construction, as illustrated in Fig. 546. The intermediate floors are generally of ¾" glass, sand blasted to prevent slippage, although 1¼" marble is occasionally used. Structural steel floor and curb angles support the floor and transfer the loads to the stack uprights.

The features of principal interest to the structural engineer in stack room work are the amounts and distribution of the loads at the lower floor, — the stacks above, with any intermediate floors, being self-supporting. For the floor construction, the following represents the usual allowance per square foot:

	L.L. = 50
¾" glass (based on net areas)	= 10
Floor framing (based on gross areas)	= 5
	T.L. = 65#/□'

or

	L.L. = 50
1¼" marble (based on net areas)	= 17
Floor framing (based on gross areas)	= 6
	T.L. = 73#/□'

The weight of books averages 22½#/c.f. and for the shelves supporting them, 7½#/c.f., or the total is 30#/c.f. A 3'-0" section with double 8" shelves occupies 30 c.f., or the total weight is 900#. Table 88 gives estimated loads for 12 tiers of 8" double stack, standard bracket type construction. The loads for other types will not vary enough to warrant any changes. The total load signifies the value at the base of each upright on the basis of ¾" floor glass and a reduction of live load of 5% for each floor from the top down. The usual number of tiers in small libraries is 3. The relative location of the end, inner, and wall uprights is shown in Fig. 544.

In a steel-framed floor which carries the loads, the beams are usually arranged 4'-6" o.c., to run parallel to the stacks and to carry the concentrations from the uprights. These may be framed into cross girders which conform to the general framing. In certain cases, such as in stone concrete slab or joist construction, the loads, if not too heavy, may be transferred by the structural floor to beams spaced in conformance with the typical framing of the building. The loads from the end and side aisles of each intermediate floor are usually carried by a wall bar, expansion bolted to the wall. In designing structural wall

TABLE 88

LOADS FOR LIBRARY STACKS*

¾" glass floors, L.L. reduced 5% for each floor from the top down

	END UPRIGHT A			INNER UPRIGHT B			WALL UPRIGHT C		
Tier	Floor Load	Shelf Load	Total Load	Floor Load	Shelf Load	Total Load	Floor Load	Shelf Load	Total Load
1st Tier		450	450 Lbs		900	900 Lbs		450	450 Lbs
2nd Tier	624 L.L. 188 D.L. 812	450	1712 Lbs	462 L.L. 140 D.L. 602	900	2402 Lbs	231 L.L. 70 D.L. 301	450	1201 Lbs
3rd Tier	593 L.L. 188 D.L. 781	450	2943 Lbs	439 L.L. 140 D.L. 579	900	3881 Lbs	219 L.L. 70 D.L. 289	450	1940 Lbs
4th Tier	563 L.L. 188 D.L. 751	450	4144 Lbs	417 L.L. 140 D.L. 557	900	5338 Lbs	208 L.L. 70 D.L. 278	450	2668 Lbs
5th Tier	535 L.L. 188 D.L. 723	450	5317 Lbs	396 L.L. 140 D.L. 536	900	6774 Lbs	198 L.L. 70 D.L. 268	450	3386 Lbs
6th Tier	508 L.L. 188 D.L. 696	450	6463 Lbs	376 L.L. 140 D.L. 516	900	8190 Lbs	188 L.L. 70 D.L. 258	450	4094 Lbs
7th Tier	483 L.L. 188 D.L. 671	450	7584 Lbs	357 L.L. 140 D.L. 497	900	9587 Lbs	179 L.L. 70 D.L. 249	450	4793 Lbs
8th Tier	459 L.L. 188 D.L. 647	450	8681 Lbs	339 L.L. 140 D.L. 479	900	10966 Lbs	170 L.L. 70 D.L. 240	450	5483 Lbs
9th Tier	436 L.L. 188 D.L. 624	450	9755 Lbs	322 L.L. 140 D.L. 462	900	12328 Lbs	162 L.L. 70 D.L. 232	450	6165 Lbs
10th Tier	414 L.L. 188 D.L. 602	450	10817 Lbs	306 L.L. 140 D.L. 446	900	13674 Lbs	154 L.L. 70 D.L. 224	450	6839 Lbs
11th Tier	393 L.L. 188 D.L. 581	450	11858 Lbs	291 L.L. 140 D.L. 431	900	15005 Lbs	146 L.L. 70 D.L. 216	450	7505 Lbs
12th Tier	373 L.L. 188 D.L. 561	450	12869 Lbs	276 L.L. 140 D.L. 416	900	16321 Lbs	139 L.L. 70 D.L. 209	450	8164 Lbs

* Courtesy of the Library Bureau, Boston, Mass.

beams, a designer strictly should allow for this in the wall load. In special cases, such as at hollow block partitions,* the side and end aisles may be cantilevered back to the uprights of the stack construction, although this is not usual. In making the structural plans, the engineer should provide for the necessary openings for stairs and lifts. In many cases, a book lift (Fig. 545) is used. The clear opening for a standard lift is $22\frac{1}{2}'' \times 29''$. The lift itself is a part of the stack contract but the openings must often occur down through to the basement to provide for the machinery which operates the lift. Stairways are required to connect the various tiers. These are of steel sub-tread construction (Art. 307) with slate treads when glass floors are used and with marble treads when marble floors are employed, and are a part of the stack contract. A typical stair run weighs about 300# and one-half of this must be supported at the foot by the structural frame. This load generally is easily provided for by a beam which is typical for the remaining frame.

* It is not desirable to expansion bolt the wall bars to hollow block walls. There may be other special circumstances when the wall treatment should be kept free

341. Framing for Fireplace Details.

In types of buildings where a series of fireplaces are to be built, such as in large apartment houses and some hotels, it is necessary to provide secondary framing of small beams and slabs in many cases, to support the flues, hearths and surrounding brickwork. Figure 547 shows typical details of this kind.

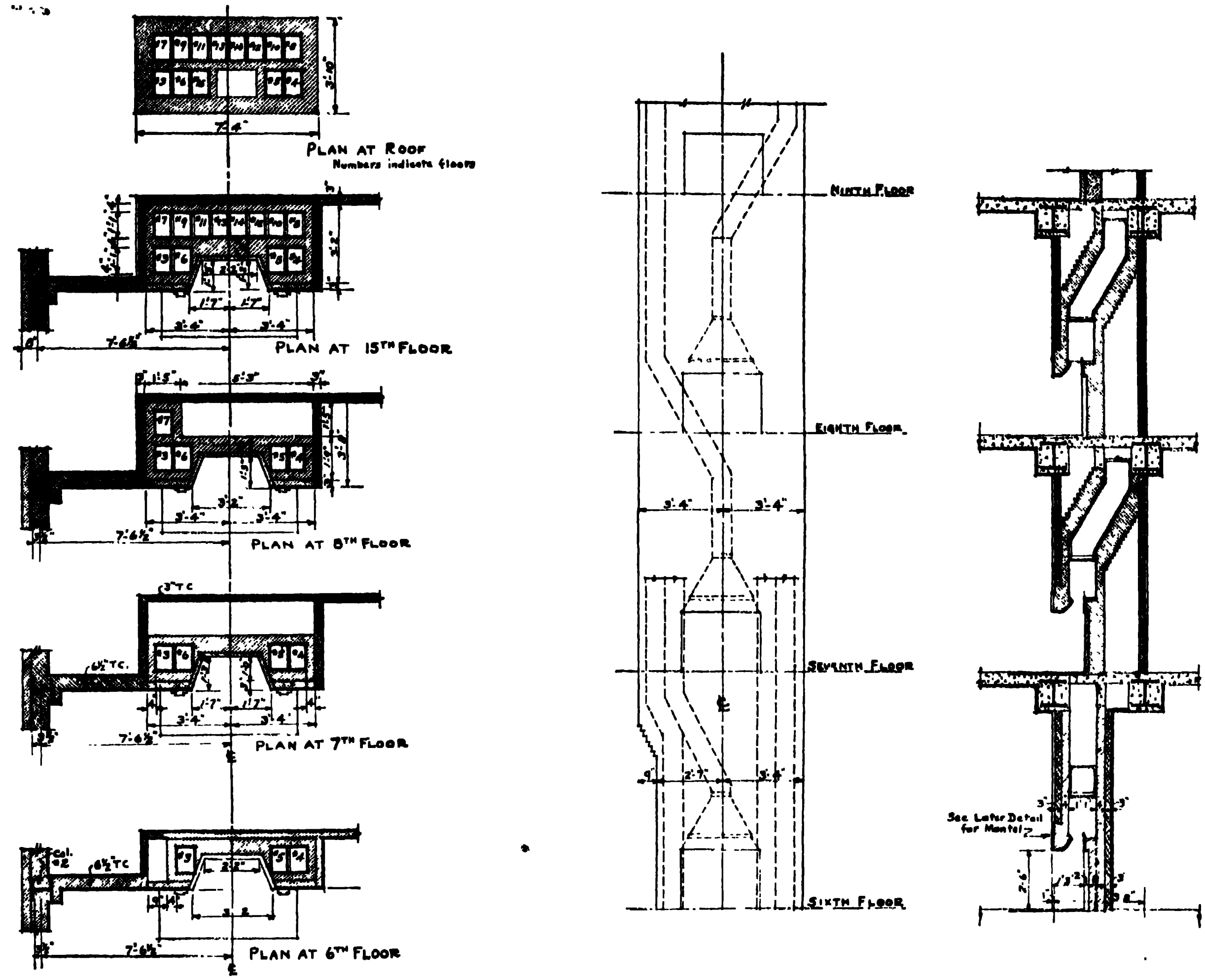


Fig. 547

The walls of a chimney should not be less than $3\frac{3}{4}''$ thick of solid brick, and usually, where several flues are involved, the walls should be of $8''$ brick. The fireplace walls should be at least $8''$ thick. None of the work should be carried on wooden floors, beams or brackets, and iron brackets or stirrups attached to wooden beams must not be used. The

flues should be as nearly vertical as possible, and in no case should the angle of their slope be greater than 45° from the vertical. Only one connection to a flue should be allowed.

Not more than two flues should occur in the same flue space, separated by brick withes, and liner joints should be split offsets. If more than two flues are involved, each third flue should be separated from the others by a smoke-tight withe or division wall not less than 3¾″ thick, bonded into the side walls. All flues for fireplaces should preferably be so separated. Concrete slabs may be used to carry fireplace hearths instead of the usual trimmer arch, provided a fill is used between the slab and the hearth.

342. Steel Water-Tanks.

As a part of the equipment, water-tanks are often installed in buildings. These may be tanks used in conjunction with sprinkler systems,* house reserve supply tanks, or hot water tanks, and so on. In connection with sprinkler systems, two types of tanks are usually provided, namely **pressure tanks** and **gravity tanks**. The best service includes tanks of both kinds. A pressure tank is generally a horizontal steel cylinder with segmental ends. Figure 548 shows tanks of this kind ready for installation.

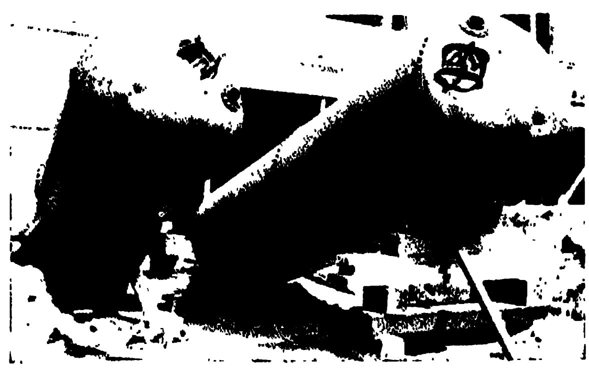


Fig. 548

A gravity tank is usually a steel cylindrical shape, placed vertically with a flat or conical top, and with a flat, hemispherical, or elliptical bottom, depending upon the type of support used. A gravity tank is set at an elevation to give the desired water pressure at the sprinkler heads in the top story. This means that the tank should be placed on the roof of the building, or else that it be made independent of the building and placed on its own tower. The latter is desirable if several buildings are to be served by the tank, and in any case, if ground space is available. However, appearances may govern this decision, and except for manufacturing buildings and structures in outlying districts, the tank is usually placed on the roof. It is located at a point where it will not interfere with penthouses, chimneys, and so on. Pressure tanks are sometimes located in the basement, or occasionally underground outside of the building, but if convenient, they should be placed under the gravity tank, as less plumbing is required.

The pressure tank serves the purpose of providing a high pressure at the start of a fire,† and the gravity tank serves as a reserve supply. Pressure tanks may be installed in two or more units. When the pressure in the pressure tank reduces to a certain amount, the water from the gravity tank flows into the pressure tank and continues the supply of water. The pressure tank must be airtight. Gravity tanks are sometimes used alone without pressure tanks, but the efficiency of the sprinkler system is diminished. Occasionally house tanks are used, which are made of excessive size, so that sprinkler service and house service may be had at the same time.

SPECIFICATION CLAUSES‡

Two independent water supplies shall be provided, at least one of which shall be automatic. Provided that, where sprinklers are required only in cellars, basements, and first stories, a connection to street main will be deemed sufficient.

Note. — Supply from street mains is not sufficient for automatic supply unless giving, in highest line of sprinklers, at least 25 pounds static pressure, and able to maintain 10 pounds pressure with the water flowing through the number of sprinklers judged liable to be opened by fire at any one time.

Pressure tanks, if used, shall have a total capacity of not less than 4500 gallons (3000 gallons of water), and in any event the tank or tanks shall contain sufficient water to supply 12½ per cent of the greatest number of sprinklers within a fire area on any one floor for 20 minutes with an average discharge of 20 gallons per minute per sprinkler.

Gravity tanks, if used, shall contain an available quantity of water sufficient to supply 25 per cent of the greatest number of sprinklers in a fire area on any floor to which it gives protection, for 20 minutes with an average discharge per sprinkler of 20 gallons per minute, but tank shall be not less than 5000 gallons available capacity.

Elevation of bottom of tank above the highest line of sprinklers on the system which it supplies shall be not less than 20 feet.

The support of gravity tanks on roofs may be provided in a number of ways. If masonry walls are used around a stair or elevator shaft, these may be employed, and if desired, the walls may be ex-

* Whether sprinkler systems are to be used or not depends upon the type of building, class of construction, funds available, and so on. In first-class buildings of public nature, they are often omitted. Where fire hazards are great, such as in manufacturing of some types, they are an investment, and usually the cost may be made up in a few years by decreased insurance rates. Sprinklers are usually required by law in second- and third-class buildings within the fire limits, especially in certain portions of the building. For a typical example of requirements, refer to "Regulations for the Installation of Gravity and Pressure Tanks" and "The Building Code" issued by the National Board of Fire Underwriters, New York City.

† The sprinklers are usually planned to release at 160° F.

‡ The Building Code of the National Board of Fire Underwriters, New York City

tended to form a tower for the tank, and in special cases, continued to house it. On a large roof it may be more expedient to use four interior columns and design them accordingly, counting the weight of the water as dead load, as well as the other actual dead load. Such columns should be of steel, encased in fireproofing materials, or of reinforced concrete. An outside wall may be employed to support a beam frame or a truss at one side, and two interior columns at the other. For narrow buildings,* trusses may be used to support the tank, spanning from one wall to the other. Figure 549 shows typical tank installations.

(a) (b)

Fig. 549

SPECIFICATION CLAUSES†

Tanks of more than 500 gallons capacity placed within any building, or on or above the roof of any building, shall be supported by steel or masonry of sufficient strength to carry the same safely. Beams shall rest at both ends on steel girders, iron or steel columns, or walls or piers of masonry.

The supporting I beams shall either have the ends built into masonry work, or shall be securely framed together in a manner to prevent possibility of overturning or buckling due to oscillation of the tank in a wind storm.

Note. — Some bad tank disasters have occurred due to neglect of this requirement.

In or near the bottom of each tank there shall be a pipe or outlet not less than 4 inches in diameter, fitted with a suitable gate valve, to permit ready drainage of the tank in case of necessity.

Wooden covers of tanks on roofs shall be covered with metal. Hoops of wooden tanks shall be of metal round in section.

Tanks having a capacity exceeding 1000 gallons and placed on or within non-fireproof buildings, shall have the supporting steel framework thoroughly encased in fireproofing material.

As far as gravity tanks themselves are concerned, special standard designs have been established by the manufacturers up to 1,000,000 gallons capacity, and in some cases, for larger capacities. Table 89 gives one set of such standard sizes.

TABLE 89‡

STANDARD SIZES--CYLINDRICAL VERTICAL STORAGE TANKS

Cap., gals.	Diam., ft. in.	Ht., ft. in.	Cap., gals.	Diam., ft in.	Ht., ft. in.	Cap., gals.	Diam., ft. in.	Ht., ft. in.
10,000	12 3	11 9	75,000	23 9	23 3	225,000	36 6	20 0
15,000	15 0	11 9	90,000	26 0	23 3	250,000	38 6	29 0
20,000	17 4	11 9	100,000	27 4	23 3	300,000	42 0	29 0
25,000	16 0	17 6	120,000	30 0	23 3	400,000	49 0	29 0
30,000	17 4	17 6	125,000	27 6	29 0	500,000	54 3	29 0
40,000	20 0	17 6	150,000	30 0	29 0	600,000	60 0	29 0
50,000	22 4	17 6	175,000	32 3	29 0	750,000	66 3	29 0
60,000	21 3	23 3	200,000	34 6	29 0	1,000,000	77 0	29 0

Of course these may be varied to suit special requirements where limited space is imperative.§

The structural engineer is principally concerned in providing an adequate support for the tanks. The capacity and corresponding diameter and height must be determined by a mechanical engineer. The heights of tanks are generally kept less than the diameters, in order to maintain a small variation in head. The size may be established from the relation that 1 cu. ft. equals 7½ gallons.¶ The weight of water is approximately 62.5# per cu. ft., so that the weight of the water, plus an allowance for the tank, may be established by the engineer. As a guide to the weight of tanks, Fig. 550 may be used to obtain average thicknesses of metal close enough for estimating weights. The following describes its use:

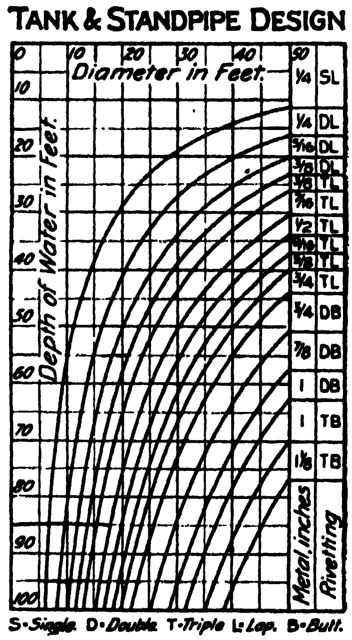


Fig. 550||

"For designing flat bottomed tanks or stand-pipes, find point of intersection of depth and diameter lines. This point will usually lie in space between two curves. Follow space upward and to the right and read thickness of metal and type of riveted joint to be used at lowest ring of shell. Follow

* An economical limit for such a case would probably be about 60′-0″.

† The Building Code of the National Board of Fire Underwriters, New York City.

‡ Chicago Bridge and Iron Works, Chicago, Ill.

§ For the theory, specifications, and examples of the design of steel stand-pipes and elevated tanks on towers, refer to Chapter XI, "Structural Engineers' Handbook" by M. S. Ketchum, — McGraw-Hill Book Co., Inc.

¶ One gallon of water = 231 cu. ins., 1728 ÷ 231 = 7.48 gallons/cu. ft., or 62.42 ÷ 7.48 = 8.34#, the weight of 1 gallon of water.

|| Pittsburgh-Des Moines Steel Tanks.

diameter line up and change thickness of metal each time said line crosses curves above.

"For metal of hemispherical bottoms, use only half the thickness given in table. A minimum thickness of $\frac{1}{4}''$ is assumed, and governs all area to the left and above top curve."*

The immediate support of a tank on a roof is usually formed by a "crib" or grillage of steel beams, or a heavy concrete slab. This in turn is transferred by girders to the columns, walls or trusses on the four corners surrounding the tank. Careful study should be made of the layout of the beams, and the top layer of beams may be cantilevered small distances from the layer underneath, to support the tank. Figure 551 shows a study of this kind.

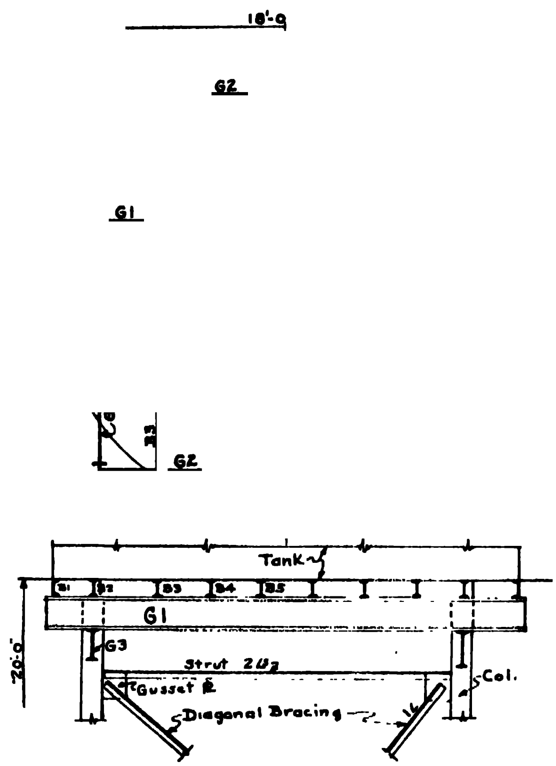


FIG. 551

The tanks are usually elevated above the roof to obtain the proper head, and generally supported above the roof plane by towers. In special cases the "crib" may be supported by the columns in the upper story of the structure, but it is generally better to use a tower, even of small height, as there are fewer indeterminate factors of resistance. The tower legs are usually sloped to give a greater spread at the bottom and a better graphical analysis. With the height of the roof and the bottom of the tank established, the difference between the two elevations will determine how much of a tower, if any, is needed.

* Pittsburgh-Des Moines Steel Tanks.

In addition to carrying the vertical loads, provision must be made against overturning due to wind when the tank is empty. The moment may be calculated from

$$M = \frac{p \cdot D \cdot H^2}{\ } \text{ in which}$$

p the wind pressure in #/□',
D the diameter of the tank in ft.,
H the height of the tank (and roof of tank if any) in ft.

The value of p is customarily taken as 30#/□' over six-tenths of the projected area, or, in other words, $30 \times 0.6 = 18$#/□' of projected area.† The uplift per linear foot of circumference of the tank, u, may be calculated from

$$u = \frac{4\,M}{D^2} \quad \frac{p}{D}. \qquad (S\text{–}89)$$

The total uplift may be obtained by multiplying u by the circumference of the tank. If a definite number of anchor bolts is fixed, the stress in each may then be calculated, and the size determined. The members carrying the anchor bolts must be designed for the upward bending induced and the leeward beams investigated for consequent increases in loads. Figure 552 shows a typical anchor bolt detail.

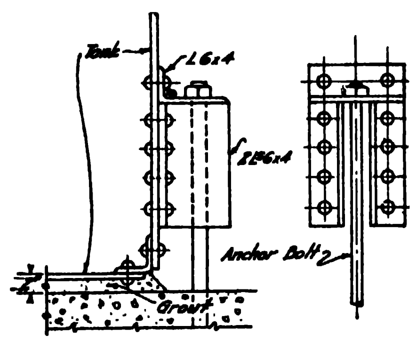


FIG. 552

When a tower is used, the legs should be tied together with diagonal sway bracing. Figure 553 shows some typical stress diagrams for theoretical loads. In actual cases, the main wind thrust is at the bottom of the tank, represented by force AB in Fig. 553. Both diagonals are used in an actual case, each being assumed to carry only the tensile stress obtained in the diagram. In a design of this kind, some engineers prefer that only the usual stresses should be used, with no allowance for combined stresses. However, many codes allow a longer combined stress, as discussed later. In addition, the wind pressure on each tower column is usually assumed as 50# per vertical foot.

† A conservative unit wind load should be used, as wind pressures on small surfaces are more nearly realized than on larger surfaces. The effective force on a cylinder is of course decreased from that on a normal surface.

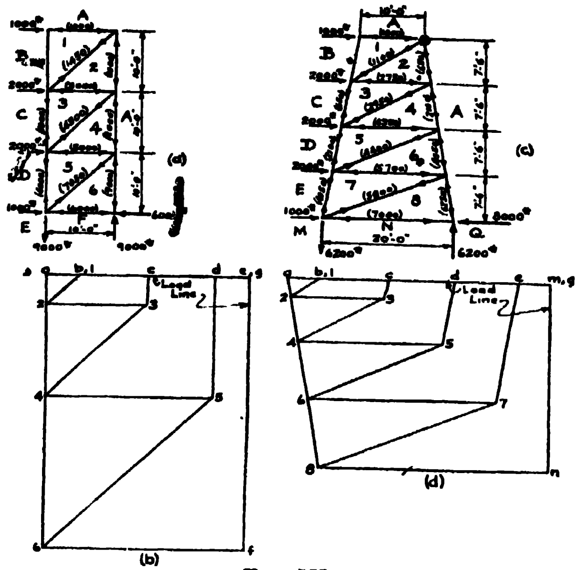


FIG. 553

Illustrative Prob. 342a. Design an arrangement to support a steel tank of 50,000 gallons capacity. Height of bottom of tank above roof 20′-0″.

From Table 89, standard size 22′-4″ diameter, 17′-6″ height.

Circumference of tank $= \pi(22.33) = 70.25'$

Area of cylinder $= 70.25 \times 17.5 = 1230\square'$

$$\text{Area of base} = \frac{\pi(22.33)^2}{4} = 392$$

$$\text{Area of top} \quad \frac{\pi(22.33)^2}{4} = 392$$

Total area $= \overline{2014}\square'$

From Fig. 550, thickness of metal $\frac{1}{4}''$.

Weight of $\frac{1}{4}''$ Pl. — 1′-0″ square $= 10.20\#$

Weight of tank $= 2014 \times 10.20 = 20{,}500$

Rivets and connections, say $= \underline{500}$

Total $= 21{,}000\#$

Weight of water $= 50{,}000 \times 8.34 = \underline{417{,}000}$

Total $= 438{,}000\#$

$$\text{Load per sq. ft. on support} = \frac{438{,}000}{392} = 1120\#/\square'.$$

The arrangement of the vertical supports under the tank should be the result of some study to obtain an economical layout. Usually supports at the corners of a square, the side of which is less than the diameter of the tank, are advantageous. This gains the advantage of allowing the top supports to cantilever small distances from the girders underneath. Columns 18′-0″ o.c. will be used.

The immediate support for the tank should be more or less uniform. Hence a series of small beams, spaced close together, or a "grillage," will be used. A 2′-6″ spacing of beams between columns will be used, with a beam 2′-5″ outside of each column at the edge of the tank, as shown in Fig. 551.

$$\text{Load on } B\text{-}1 = \frac{1120 \times 2.42}{2} = 1360\#/\text{ft.}$$

Beam $= \underline{10}$

1370

$$M = 1.5 \times 1370 \times (6)^2 = 74{,}000''\#$$

$$\frac{I}{c} = \frac{74{,}000}{16{,}000} = 4.63''^3 \qquad \text{Use 7 [9.8.}$$

Load on supported span of interior beams

$1120 \times 2.5 = 2800\#/\text{ft.}$

Beam $= \underline{20}$

2820

$$M = 1.5 \times 2820 \times (6)^2 = 152{,}000''\#$$

$$\frac{I}{c} = \frac{152{,}000}{16{,}000} = 9.5''^3 \qquad \text{Use 7 I 15.3.}$$

Cantilever portion = 2′-0″ maximum (scaled)

$$M = \frac{w \cdot L^2}{2} = \frac{2820 \times (2)^2}{2} = 5640'\# = 67{,}700''\#$$

7 I 15.3 ample.

Concentration on G-1

$2820 \times 6 = 16{,}920\#.$

With the spacings and loads, the bending moment and the required size for G-1 may be calculated in the usual way. The influence of the load on the projection beyond G-3 should be included, of course. Similarly, the loads on G-2 may be calculated and the size determined, and when the reactions from G-1 are known, the size required for G-3 may be calculated. With the values of the reactions of G-2 and G-3 established, the vertical load on a column may be computed.

In addition, there is a vertical force exerted on the columns due to wind action.

$$\text{Pressure on tank} = 18 \times 22.33 \times 17.5 = 7030\#$$

$$\text{Overturning moment} = 7030 \times \frac{17.5}{2} = 61{,}500'\#.$$

The worst effect is when the wind is blowing in a direction at 45° with the column center-lines. Assume that in such a case one column receives all the uplift.

Diagonal distance $= 1.414 \times 18.0 = 25.4'$

$$\text{Uplift} = \frac{61{,}500}{25.4} = 2420\#.$$

This force must be provided against by anchorage at the top of the column to hold the tank on, through connection of the tank to the beams and of the beams to the columns. On the corner opposite to where the uplift is exerted, a downward force of 2420# is exerted on the column. This should theoretically be added to the column load. If such a value is less than 50% of the load on a column due to the weight of the tank and water, it may be neglected, and the columns proportioned for the direct load with the usual allowable stress. In other words, combined stresses due to wind action may be increased 50% above the usual allowable in accordance with many specifications.

At the bottom of the columns, a greater uplift occurs, due to the greater leverage of the wind.

Pressure on tank (as above) $= 7030\#$.

$$\text{Arm to foot of column} = 20 + \frac{17.5}{2} = 28.75'.$$

Moment $= 7030 \times 28.75 = 202{,}000'\#$.

Wind on column assumed as 50#/ft.

Wind force on column $= 50 \times 20 = 1000\#$.

Arm to foot of column $= 10'$.

Moment $= 1000 \times 10 = 10{,}000'\#$.

Total moment $= 202{,}000 + 10{,}000 = 212{,}000'\#$.

$$\text{Uplift} = \frac{212{,}000}{25.4} = 8340\#.$$

The connection at the foot of each column should be strong enough to resist such a force and such a value should also be added into the column load to design the support under it for compression.

The stresses in the diagonal sway bracing and bottom struts may be determined in a manner similar to that illustrated in Fig. 553. For the sake of stiffness, it is better to frame the members G-2 into the column rather than run them over the top.

The tank columns may be supported by columns of the building directly under them or, in the case of an isolated tank, on footings. The columns are sometimes made of four angles laced in order to obtain stiff sections. They should preferably be given an outward slope, as a better spread is obtained at the bottom.

In special cases, free-standing, elevated, water-tanks are used, where there is available space on the lot, and the appearance is not objectionable, such as in some mill building projects. Figure 554 shows typical characteristics and Fig. 555 a stress diagram for such types. These tanks may have either hemispherical or elliptical bottoms. The former is more common and has the greater capacity for a given amount of metal. The wind pressure on the tower is usually taken as 200# per vertical ft., or 50# on each tower column. The maximum column wind load is obtained by multiplying the values in the stress diagram by $\sqrt{2}$. The column is designed for one-quarter of the weight of the tower, tank, and water, plus that portion of the maximum wind loads in excess of one-quarter of the dead load. Table 90 shows some standard sizes.

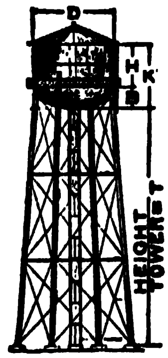


Fig. 554*

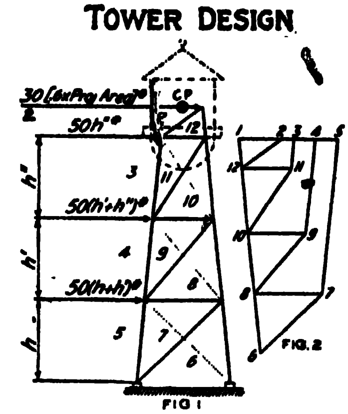


Fig. 555†

The size of the space required for the footings should be about 5′-0″ greater than the square of the base on each side in each direction. The structural engineer is usually only involved in providing proper foundations. The design of the balance is usually a standard one made by the manufacturer.

TABLE 90*

ELLIPTICAL AND HEMISPHERICAL BOTTOM ELEVATED TANKS

Capacity, gals.	Elliptical bottom, ft., in.			Hemispherical bottom, ft., in.		
	D	H	K	D	H	K
15,000	15 0	9 0	12 9	12 0	14 0	18 6
20,000	16 0	11 0	15 0	12 9	17 3	22 1½
25,000	17 6	11 0	15 4	14 1	17 3	22 9½
30,000	18 6	12 0	16 7	15 3	17 3	23 4½
35,000	19 0	13 4	18 1	16 4	17 3	23 11
40,000	20 0	13 9	18 9	17 4	17 3	24 5
50,000	22 0	14 0	19 6	19 0	17 6	25 6
60,000	24 0	14 0	20 0	19 0	22 3	30 3
70,000	25 0	15 0	21 3	21 0	20 3	29 3
75,000	26 0	15 0	21 6	22 0	19 4	28 10
80,000	26 0	16 0	22 6	22 0	21 1	30 7
100,000	28 8	16 0	23 2	22 0	28 0	37 6
125,000	28 8	21 2	28 4	24 0	29 0	39 6
150,000	34 0	16 6	25 0	26 0	29 3	40 9
200,000	38 0	17 6	27 0	28 0	34 6	47 0
250,000	40 0	20 0	30 0	30 0	37 2	50 8
300,000	41 0	23 9	34 0	32 0	40 0	54 6
400,000	47 0	23 2	34 11	35 0	44 0	60 0
500,000	51 0	24 3	37 0	38 0	46 6	64 0

Dimensions of Standard Tanks:

Elliptical. Depth of bottom $B = \dfrac{D}{4}$

Square of base $= .71\,D + .118\,(T + B)$

Hemispherical. Depth of bottom $B = \dfrac{D}{2}$

Square of base $= .71\,D + .162\,(T + B)$

* Chicago Bridge and Iron Works.
† Pittsburgh-Des Moines Steel Tanks.

TABLE 91

DATA FOR TANKS

Axis Horizontal
Contents per Ft. of Length

Diameter of Tank (Ins.)	Two-Thirds Full		Full	
	Gallons	Cu. Ft.	Gallons	Cu. Ft.
36	37 4	5 01	52 9	7 07
42	50 9	6.81	71 9	9 62
48	66 6	8 90	94 0	12 57
54	89 5	11.95	119 0	15 90
60	104 0	13 90	146.9	19 62
66	125 9	16 82	177 7	23 75
72	149 8	20 02	211.5	28 27
78	175 8	23 50	248 2	33 18
84	195 5	26 14	287.9	38 48

Axis Vertical
Contents per Foot of Depth

Dia. Ft.	Gal.	Cu. Ft.	Dia. Ft.	Gal.	Cu. Ft.	Dia. Ft.	Gal.	Cu. Ft.	Dia. Ft.	Gal.	Cu. Ft.
3	52 88	7 069	10	587.5	78 54	23	3108	415.5	37	8041	1075
3½	71.97	9 621	10½	647.7	86 59	24	3384	452.4	38	8482	1134
4	94 00	12 57	11	710.8	95 03	25	3672	490.9	39	8930	1195
4½	119 0	15 90	12	846.0	113 1	26	3971	530.9	40	9402	1257
5	146 9	19 63	13	992.6	132 7	27	4283	572.6	41	9873	1320
5½	177 7	23 76	14	1151	153 9	28	4606	615.8	42	10360	1385
6	211 6	28.27	15	1322	176 7	29	4940	660.5	43	10811	1452
6½	248 2	33 18	16	1504	201 1	30	5287	706.9	44	11377	1521
7	287 9	38 48	17	1698	227.0	31	5646	754.8	45	11893	1590
7½	330 5	44 18	18	1903	254 5	32	6015	804.2	46	12432	1662
8	376 0	50 27	19	2120	283 5	33	6398	855.3	47	12978	1735
8½	424 5	56 75	20	2350	314 2	34	6791	907.9	48	13539	1810
9	475 8	63 62	21	2591	346 4	35	7196	962.1	49	14107	1886
9½	530.2	70 82	22	2843	380 1	36	7615	1018	50	14683	1963

Pressure tanks are designed according to the theory of hoop tension.* They are generally planned to be used ⅔ full of water so that when the required capacity of the tank is known, it should be multiplied by 1½ to establish the volume of the tank. Table 91 gives data for tanks.

Usually the thickness of metal is ¼″ (minimum) so that, with the size known, the weight of the tank and the water may be calculated (see Illustrative Prob. 342a). The thicknesses of shell required for the cylinder and its ends are usually determined by the mechanical engineer. The necessary riveting to develop the plates is determined by the tank fabricator. The tank is generally supported by cast-iron saddles placed under the quarter-points of the length of the tank. The saddles may be supported by a pair of beams.

"House" tanks are used as reserve supplies, especially in hotels and large apartment houses. These are usually of 2000 gallon units, and cylindrical in shape, placed horizontally. They are used to give a water supply if the public service is cut off, or in cases where the public supply lacks sufficient pressure to reach the upper floors. These are sometimes located in elevator penthouses. The weights may be calculated in a manner similar to that for other tanks, and a method of support planned accordingly. The strength of the tank is made sufficient to span between saddles.

Prob. 342b. Design an arrangement similar to Fig. 551 for a steel tank of 40,000 gallons capacity. Height of bottom of tank above roof 14′-0″. Use 16′-0″ spacing of columns at top and attach to building columns 18′-0″ o.c. at bottom. Give sizes of diagonal bracing and struts as well as other members.

Prob. 342c. Design a free-standing steel tank of the type shown in Fig. 554, hemispherical bottom, 60,000 gallons capacity. Use concrete bases under columns and give size of anchor bolts. Tower 60′-0″ high.

343. Interior, Steel Smokestacks.

SPECIFICATION CLAUSES†

> The smoke flue of every high pressure steam boiler and every appliance producing a corresponding temperature in the smoke flue shall, if built of brick, stone, reinforced concrete or other approved masonry, be lined on all sides with not less than 4 inches of fire brick laid in fire mortar for a distance of at least 25 feet from the point where the smoke connection of the boiler enters the flue.
>
> Interior vertical smokestacks or flues for steam boilers or other furnaces, and similar heating devices producing a corresponding temperature, may be of metal not less than No. 10 U. S. gauge, properly riveted, jointed, and braced at intervals of at least 20 feet. Such stacks shall either be enclosed by approved masonry walls not less than 8 inches thick with an air space of at least 4 inches between lining and wall; or if such stacks or flues are not enclosed with masonry they shall have a clearance from all combustible material of not less than one-half the diameter of the stack, but not less than 24 inches, unless the combustible material be properly guarded by loose fitting metal shields, in which case the distance shall be not less than 12 inches. Where such a stack passes through a wooden framed roof, it shall be guarded by a galvanized iron ventilating thimble extending from at least 9 inches below the underside of the ceiling or roof beams to at least 9 inches above the roof, and the ventilating thimble shall have a clearance of not less than 18 inches, except that for stacks for low grade furnaces such as hot air, hot water, and low pressure steam heating furnaces, coffee roasting ovens, candy furnaces, etc., the clearance may be reduced to 12 inches. Metal smokestacks shall not be permitted to pass through floors. Smoke flues shall not be permitted inside of vent flues for ranges.

In office buildings and the like, the boilers are usually connected to steel stacks. These vary from 12 to 50 square feet in area, and are circular, elliptical or rectangular (with rounded corners) in shape, depending upon the size of the heating plant, and the general location respectively. They are made up of steel plate material with riveted joints and connecting angles. The stack may be placed outside of the building in certain instances, but more usually it is located inside the structure, principally for architectural reasons, but also to eliminate the high cost of maintenance.

Stacks may be built as self-supporting for their full height and carried on a grillage or a cast base, placed on a separate concrete footing, or the stack may be supported at intervals of its height by the structural frame. For the second type, the stack is shipped to the job in two-story lengths and joined by field riveting the flange angles which are attached to the ends. The stack is braced at alternate floors by abutting angles which are connected to the floor beams. These bracing angles are not connected to the stack so that the latter may expand or contract without throwing stress into the angles. When the stack is to be supported by the structural frame, it is usually carried at every second floor. Angle brackets which rest on the floor beams are riveted to the stack, as shown in Fig. 556. **The stack is more commonly supported by the structural frame** than made self-sustaining. More steel is required for the self-sustaining stack, and the erection of the

* The thickness of the plates in the shell may be calculated from $\frac{p \cdot r \cdot F}{\ }$, and the thickness of the plates in the heads may be calculated from $t_h = \frac{p \quad \cdot F}{0.6\, s}$, in which r = the radius of the cylinder in ins., t = the thickness of the plate in ins., F = the factor of safety, s = the ultimate strength of the steel in #/□″, and E = the efficiency of the longitudinal seams. The radius of the dished head is usually assumed equal to the diameter of the tank cylinder.

† The Building Code of the National Board of Fire Underwriters, New York City.

supported stack may start at any logical floor. Field riveting is thus reduced to a minimum.

Stacks are generally designed by the heating engineer, and fabricated and erected by a contractor making a specialty of such work. A hood is placed on the stack above the roof line to prevent the entrance of rain and snow, and the usual clean-out door is provided in the basement, as well as the opening for the connection of the breeching from the

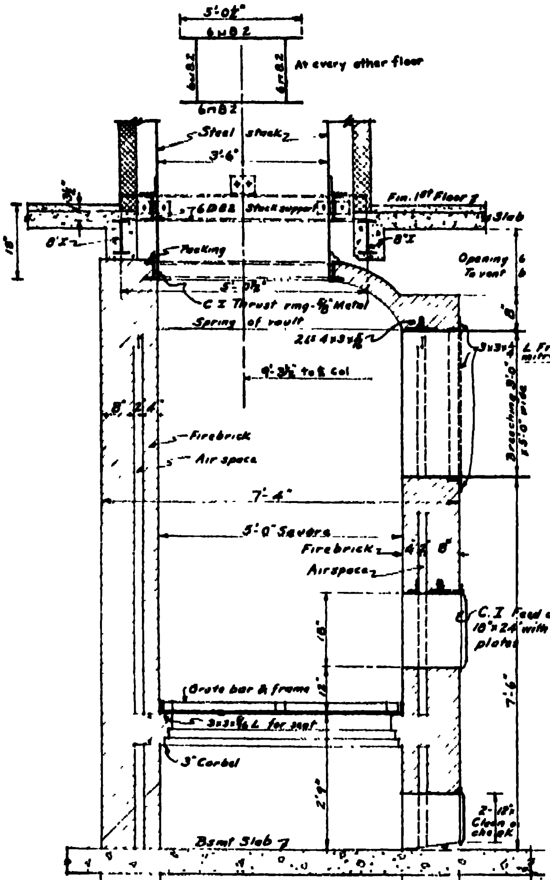


FIG. 556

boilers. In some cases, the stack in the basement is built of brick with a brick fire-lining and air-space. This requires a special cast-iron ring under the first floor to support the steel stack above. The support of the brick walls in such a case is involved in the planning of the foundations. In some instances, the lower portion of the stack is provided with a grating and used as an incinerator.

When an interior stack is planned for a building, the structural engineer is interested in only certain features in connection with it. The first point is to determine whether the stack must be supported by the structural frame or whether it is to be self-supporting. In the latter case, the stack footing may affect the arrangement of the other foundations. If the stack is to be supported by the frame, the structural engineer is interested to obtain the loads which are to be brought on to the beams and to arrange supporting channels framed between or on top of the beams, such as shown in Fig. 556. The weight of the various sections of the stack may usually be obtained from the heating engineer. Stacks supported at alternate floors are usually of $\frac{5}{16}''$ plate. If the diameter is known, the approximate weight to be carried may be determined on this basis. Stacks are often lined* for a portion or the full extent of their height, depending upon conditions.

Illustrative Prob. 343a. Determine the load to be applied to a given beam, due to a steel stack, if the stack is to be supported at four points. Stack 6′-0″ diameter, $\frac{5}{16}''$ plate. Story height 10′-6″. What will be the load at a supporting point in the lower stories if the stack is to be lined in the lower portion of its length?

Circumference of stack $\dfrac{\pi(6)^2}{4} = 28.27'$.

Length supported $= 2 \times 10.5 = 21.0$.
Surface $= 21.0 \times 28.27 = 594\square'$.
Steel plate $= 12.75\#/\square'$.
Weight $= 12.75 \times 594 = 7580\#$.

Load at one supporting point $= \dfrac{7580}{4} = 1880\#$.

Lower portion:

Volume asbestos $= 28.27 \times \frac{2}{12} \times 21.0 = 99$ cu. ft.
Asbestos lining (say $30\#$/cu. ft.) $= 30 \times 99 = 3000\#$.
$\frac{21}{3} = 7$ shelf angles. Wt./ft. of $2\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$ L $= 2.98\#$
$7 \times 28.27 \times 2.98 = 590\#$.
$7580 + 3000 + 590 = 9170\#$.
Load at one supporting point of lower portion

$$= \frac{9170}{4} = 2300\#.$$

344. Exterior Steel Stacks.

In certain types of structures, exterior stacks are used. These may be built of steel, radial brick or concrete.† Steel stacks are used principally when economy is the motive, or when the stack is to be carried on the boiler breeching. They are from 30% to 50% cheaper than masonry chimneys, although their appearance is not as good. The design of steel stacks is quite well standardized by companies making a specialty of such work. However, it is sometimes convenient for a structural engineer to understand the basic principles of such design, either in checking work or in making up preliminary estimates.

* A common lining consists of 2″ asbestos sheeting supported by internal rings of $2\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$ angles (with $1\frac{1}{2}''$ leg horizontal) 3′-0″ apart, riveted to the stack. Open holes through which dowels are inserted to hold the asbestos in place are punched in the outstanding legs, and all joints are pointed up with a cement plaster.

† For a discussion of radial brick and concrete chimneys, refer to "A Treatise on Masonry Construction" by Ira O. Baker, and "Principles of Reinforced Concrete Construction" by F. E. Turneaure and E. R. Maurer, — John Wiley and Sons, Inc., Publishers.

Steel stacks may be of two types, — either the **guyed** or the **self-sustaining.** The first may be used for small installations, in cases where the stack is to be supported on the breeching and the boiler setting is used as the chimney foundation, or where it is not feasible to use a separate base as large as would be required for sufficient anchorage and stability in itself. Self-sustaining steel stacks are more commonly employed, as they are more sightly, and no guy wires need to be extended to points perhaps not altogether convenient. Less space is required, they are easier to construct and to erect, they weigh less, and less area is exposed to wind action. Larger sizes (greater than 4'-0" diameter) are generally made self-sustaining.

Steel stacks are preferably lined to ⅓ their height with 4" fire brick, with a 1" space between it and the stack filled with sand. This provides for expansion, protects the inside from corrosion, prevents radiation, and provides a better draft. The lining is not counted upon for strength.

The thickness of the shell may be determined from the following relations:

$$P \cdot h = s \cdot \frac{I}{c}$$

$$= \frac{s \cdot \pi}{32}\left(\frac{D^4 - d^4}{D}\right), \text{ in which}$$

P = the total wind pressure above any given plane, in lbs.,

h = the height to the center of the wind pressure above the given plane, in ins.,

D = the outside diameter of the stack in ins., and

d = the inside diameter of the stack in ins.

$$t \quad D - d, \text{ in ins.}$$

The tendency toward failure would be by flattening and bending rather than by rupture, so that a low working stress should be used. The value of s is usually taken as 6000#/□" for single riveted joints, and as 8000#/□" for double riveted joints. The thickness should not be made less than minimum requirements as given by codes. For very large stacks, a ¼" minimum should be used.

SPECIFICATION CLAUSE*

Exterior metal smoke flues for boilers, large cooking ranges, and similar heating devices, shall be of approved construction and supported on approved masonry foundations, and shall have a clearance of at least 4 inches from the outside wall. Such flues having an area not exceeding 255 square inches shall be constructed of not less than No. 16 U. S. gauge metal; if the area exceeds 255 square inches the thickness of the metal shall be not less than No. 10 U. S. gauge.

* The Building Code of the National Board of Fire Underwriters, New York City.

Illustrative Prob. 344a. Determine the thickness of stack required for a section 150'-0" down from the top, if the outside diameter is 12'-0".

In circular chimneys, the area subjected to wind pressure may be assumed as 60% of the diametral area.

$$30 \times 0.6 = 18\#/\square'.$$

$$\text{Wind load} = 150 \times 12 \times 18 \quad 32{,}400\#.$$

$$\text{Arm} = \frac{150}{2} \times 12 = 900''.$$

$$32{,}400 \times 900 \quad \frac{8000 \times \pi}{32}\left(\frac{144^4 - d^4}{144}\right).$$

$$d = 143.42''.$$

$$t = 144.00 - 143.42 = 0.29''. \qquad \text{Use } \tfrac{5}{16}''.$$

An approximate solution may be had by the use of $p \quad \frac{M}{0.8\ D^2}$ in which p the stress per linear inch. For the above data,

$$p = \frac{32{,}400 \times 900}{0.8 \times (144)^2} = 1770\#,$$

$$t = \frac{1770}{8000} = 0.23''.$$

In guyed stacks, either one or two sets of guys may be used. Usually one set is sufficient, placed at ⅔ the height, with 4 guys in the set equipped with turnbuckles. Figure 557 gives a typical illustration

FIG. 557

of a small stack. An approximate solution for the size of the guy ropes is based on the assumption that one guy takes all the horizontal force of the wind on an assumed tributary area. This area is assumed as that above the point of attachment plus 0.6 of the area below. The guys are usually placed at 45° inclinations. Then H (the horizontal wind force calculated as above) divided by sin 45° is the stress in the guy rope. An initial tension of 2500# to 4500# should be assumed according to the size of the rope.

Illustrative Prob. 344b. What size of guy rope should be used for a 4'-0" diameter stack, 150'-0" high?

Guys attached at $\frac{2}{3}$ H = 100′-0″. 150 − 100 50′.
50 + 0.6 × 100 = 110′ tributary height.
110 × 4 = 440□′ tributary area.
440 × 18 = 7900# wind force.

Use 45° slope. $\frac{7900}{0.707}$ = 11,200#.

Initial tension 4300

15,500#. Use $\frac{3}{4}$″ wire rope.

Strengths of wire rope may be obtained from manufacturers' catalogues.

The guy ropes are attached to a steel band which is clamped to the stack. The lower ends must be anchored at convenient points and with sufficient embedment to resist the tension.

Steel stacks are riveted together in sections, and for small lengths (up to say 80′-0″, two-car lengths) may be shop riveted. The usual rules for riveting are generally followed as to pitches, edge distances and so on, and $\frac{1}{2}$″ rivets are the minimum size used.

The base is usually belled out to a diameter of $1\frac{1}{2}$ to 2 times the diameter of the stack and the height of the bell is usually the same as the bottom diameter. It should have a straight slope, as a cone possesses the greatest strength. This bell is attached to a heavy cast-iron base plate, usually made in six sections bolted together. The size of the base plate is determined from the allowable pressure under it. Anchor bolts are connected to the base plate and embedded in the foundation. The anchor bolts must be of sufficient size to resist the uplift due to wind moment (Art. 343).

Prob. 344c. Check the thickness of metal for the following conditions for a self-sustaining steel stack.

Inside diameter at top = 6′-10″. Height 164′-0″.
Fire-brick lining first 100′-0″ of height.

Top section (45′), $\frac{1}{4}$″ plate; next 45′ below, $\frac{5}{16}$″ plate; next 45′ below, $\frac{3}{8}$″ plate; remaining 29′, $\frac{7}{16}$″ plate. Smoke connection, 12′-0″ ellipse, 20′-0″ above ground. Flare at base, 14′-9″ outside diameter. Footing 22′-0″ square at top, 26′-0″ square at bottom, 9′-0″ deep. Determine size of anchor bolts, if 12 are used, assuming one-half of them resist uplift at any one time. What size of cast-iron base is required if the allowed pressure on the concrete is 500#/□″?

Prob. 344d. Design a guyed steel stack to have a diameter of 6′-0″ and to be 160′-0″ high.

345. Openings for Mail Chutes.

Where mail chutes are to be installed, a flat, vertical, continuous opening through the various stories is required. This means that openings must be left in the floor construction, as the chute must be left open and exposed. An opening 6″ wide and 12″ long in which iron thimbles are set must be provided. In fire-resisting construction, a pair of 2 × 2 × $\frac{1}{4}$ angles are set at the back of the opening, one in each rear corner.

346. Sky-signs.

SPECIFICATION CLAUSES*

All other signs or billboards within the fire limits shall be entirely constructed of incombustible materials, including all supports and braces for same.

Any letter, word, model, sign, device or representation in the nature of an advertisement, announcement or direction, supported or attached wholly or in part over or above any wall, building or structure, shall be deemed to be a sky-sign. Except as herein specified sky-signs shall be constructed entirely of metal, including the supports and braces for same, and no sky-sign shall project beyond the building line.

Within the fire limits no sky-sign shall be supported, anchored or braced to the wooden beams or other framework of a building which is over three stories high.

No sign attached to the side of a building or structure fronting upon a public thoroughfare, shall project more than 5 feet outside the building line.

Sky-signs shall be set back at least 8 feet from the cornice or wall on a street front, shall not project more than 25 feet above the roof of a building, and shall have a space at least 6 feet in height between the bottom of the sign and the roof.

All such signs shall be designed to withstand a wind pressure of at least 30 pounds per square foot of surface.

No sign or billboard shall be so constructed as to obstruct any door, window or fire-escape, on any building.

Before the erection of any sign or billboard shall have been commenced, a permit for the erection of the same shall be obtained from the Superintendent. Each application for the erection of any sign or billboard shall be accompanied by a written consent of the owner or owners, or the lessee or lessees of the property on which it is to be erected.

This section shall apply to all signs hereafter erected whether placed upon new or existing buildings.

The design of the frames for sky-signs generally involves only the principles of small steel truss design (Chap. 16). It should be remembered that the stresses must be determined on the basis of the wind blowing in either direction. The wind pressure of 30#/□′ usually specified is considered as being applied over the whole area of the sign, regardless of openings. The details for any particular case will of course depend upon the type of sign and the surrounding conditions. It is very important in such cases to check the roof frame for reverse bending due to brace reactions.

347. Truck Loads.

In planning loading platforms, shipping spaces, coal pockets, and the like, it is wise to check up the framing for the most unfavorable conditions resulting from auto trucks. The following represents good practice for such loads:

* The Building Code of the National Board of Fire Underwriters.

SPECIFICATION CLAUSES*

Members shall be proportioned to carry a 20-ton auto truck having 6 tons on one axle and 14 tons on the other axle, the axles being 12 feet apart and the distance between wheels 6 feet. This truck is assumed to occupy a floor space 32 feet in length and 10 feet in breadth, overhanging all wheels an equal amount.

Where plank floors or floors resting on planks are used, each wheel load of the auto truck may be considered to be distributed over a width of floor equal in feet to the thickness in inches of the supporting layer of planking. The width over which the load is distributed shall never, however, be taken as more than 6 feet.

Where solid floors are used, each wheel load may be assumed to be distributed over a width of 6 feet.

For flooring no impact need be considered.

For stringers or floor beams, 50% for impact shall be added.

The use of the above would apply to special isolated cases. For garages a uniform live load of 150#/□′ is generally used (Art. 89). Referring to the above, 20 tons = 40,000#, divided by 32′ × 10′ = 320′, is only 125#/□′.

Heights of doors to accommodate all kinds of trucks should be 14′-0″. However, all except the very largest are 11′-0″ high, so that 12′-0″ doors are often sufficient. A width of 9′-0″ should be the minimum for doors. The following represents **average data** for automobiles:

	Trucks	Touring Cars
Length	24′-6″	17′-3″
Width	8′-4″	5′-10″
Front Overhang	3′-0″	1′-11″
Wheel Base	14′-6″	11′-10″
Rear Overhang	7′-0″	3′-6″
Tread — Front wheels	5′-0″	4′-8″
Tread — Rear wheels	5′-6″	4′-8″
Radius of Clearance Circle	30′-6″	30′-3″

348. Track Loads.

Occasionally in the planning of industrial buildings, it is necessary to provide sidings from railroad tracks. In some cases, these may not be on the ground, and they may have to be supported by framing. The weights of actual rolling stock of railroads are so variable that typical loads are used. The most common of these are the so-called Cooper's loads. These correspond to two consolidation locomotives, followed by a uniform train load representing the weight of the heaviest cars.† Figure 558 shows the loads for Cooper's E–50, in which the values are **axle loads.** The E–50 is classed as standard, and E–45, E–40 and E–30 combinations are called light traffic, and the E–55 and E–60, heavy traffic. All loadings are based on the same wheel spacing, and the loads are proportional to the ratio of the index number to 50. Thus for E–40 loading, the wheel loads are $\frac{4}{5}$ of those for E–50, and so on. The tendency of the railroads is toward the use of heavier locomotives, so that usually E–50 or E–60 loadings are used, depending upon the probable type of service that will be employed.

The use of the above loads involves employing several criteria for maximum moments and shears‡ and the tendency, particularly in recent practice, even for the smaller railroad structures, is to use **equivalent uniform loads.** The latter values are sufficiently accurate for the usual cases. For instance, the moment is computed from the criterion for maximum bending for a given span for the actual loads

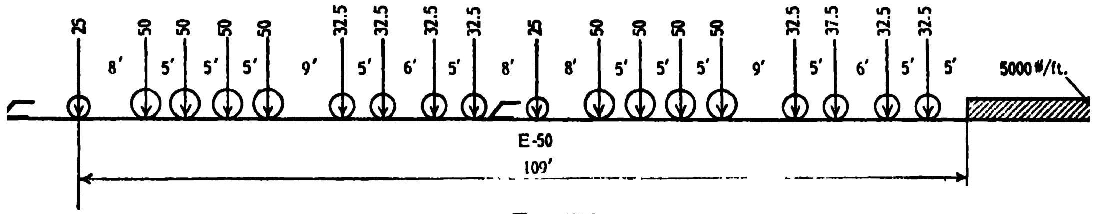


FIG. 558

and from this, the load per foot which will produce the same maximum moment at mid-span is calculated. Similar computations are made for the maximum shear. Table 92 gives such values. It should be noted that the equivalent uniform load for a given span is not the same for maximum moment as for maximum shear.

Values for Cooper's E–50 loading may be obtained by multiplying the tabulated values by $1\frac{1}{4}$, for E–60, by $1\frac{1}{2}$, and so on.

Impact should be included in addition to the loads. The following formula is quite common for railroad work:

$$I = P\left(\frac{300}{L + 300}\right), \text{ in which} \qquad (S\text{–}90)$$

I = the impact stress,

P = the stress due to the loads tabulated, and

L = the length of the loaded span, in feet.

For cases of sidings, and the like, trains will not be run at high speeds probably, so that an impact allowable of 50% is generally sufficient.

* Department of Public Utilities of Massachusetts. These specifications are given for highway bridges but may be used for special cases in buildings in the absence of other recommendations.

† A Pennsylvania R. R. coal car has a capacity of 140,000# and a total weight of 158,200#.

‡ See any standard text book on mechanics.

In planning arrangements for the accommodation of tracks, clearances must be maintained. Figure 559 shows some standard clearances, although these vary somewhat with different railroads. If the track is on a curve, additional clearance must of course be provided.

TABLE 92*

MAXIMUM MOMENTS, SHEARS, FLOOR BEAM REACTIONS AND EQUIVALENT UNIFORM LOADS FOR COOPER'S CLASS E-40 FOR ONE TRACK

Span Ft.	$M_{max.}$ Ft.-lbs.	Max. End Shear #	Max. F.B. React. #	Equiv. Unif. Ld. #/ft. For M	For V	For R
10	112,500	60,000	80,000	9000	12,000	8000
11	131,400	65,500	87,300	8690	11,910	7940
12	160,000	70,000	93,300	8890	11,670	7770
13	190,000	73,800	98,500	9000	11,350	7580
14	220,000	77,200	104,300	8980	11,030	7450
15	250,000	80,000	109,300	8890	10,670	7290
16	280,000	85,000	113,700	8750	10,620	7110
17	310,000	89,500	117,600	8580	10,530	6920
18	340,000	93,400	121,300	8470	10,380	6740
19	373,200	96,800	125,800	8270	10,190	6620
20	412,500	100,000	131,100	8250	10,000	6560
25	610,000	113,600	151,300	7840	9090	6050
30	821,000	126,100	172,500	7300	8410	5750
35	1,046,000	138,400	195,200	6840	7910	5570
40	1,311,000	150,800	.....	6560	7540	....
50	1,902,000	174,200	.....	6090	6970	....
60	2,599,000	195,200	....	5780	6510	....
70	3,415,000	221,000	.....	5580	6310	...
80	4,321,000	248,400	.....	5400	6210	...
90	5,341,000	274,500	.....	5280	6100	....
100	6,440,000	300,000	....	5150	6000	....
125	9,993,000	350,800	.....	5120	5740	....

* Merriman's Civil Engineer's Pocketbook, John Wiley and Sons, Inc., Publishers, New York City.

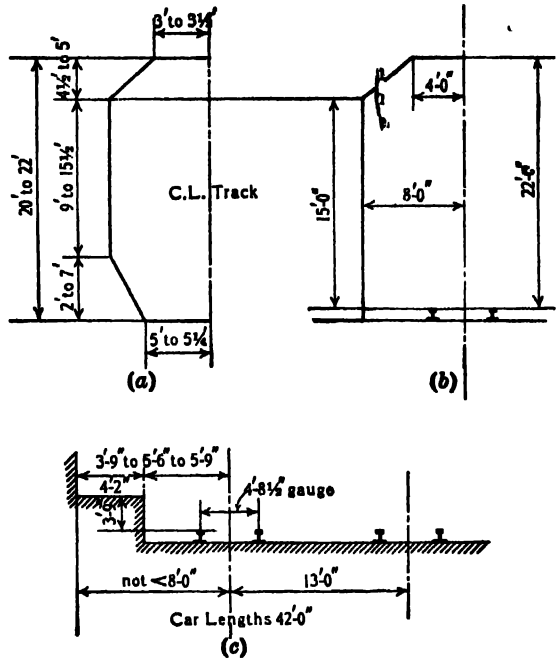
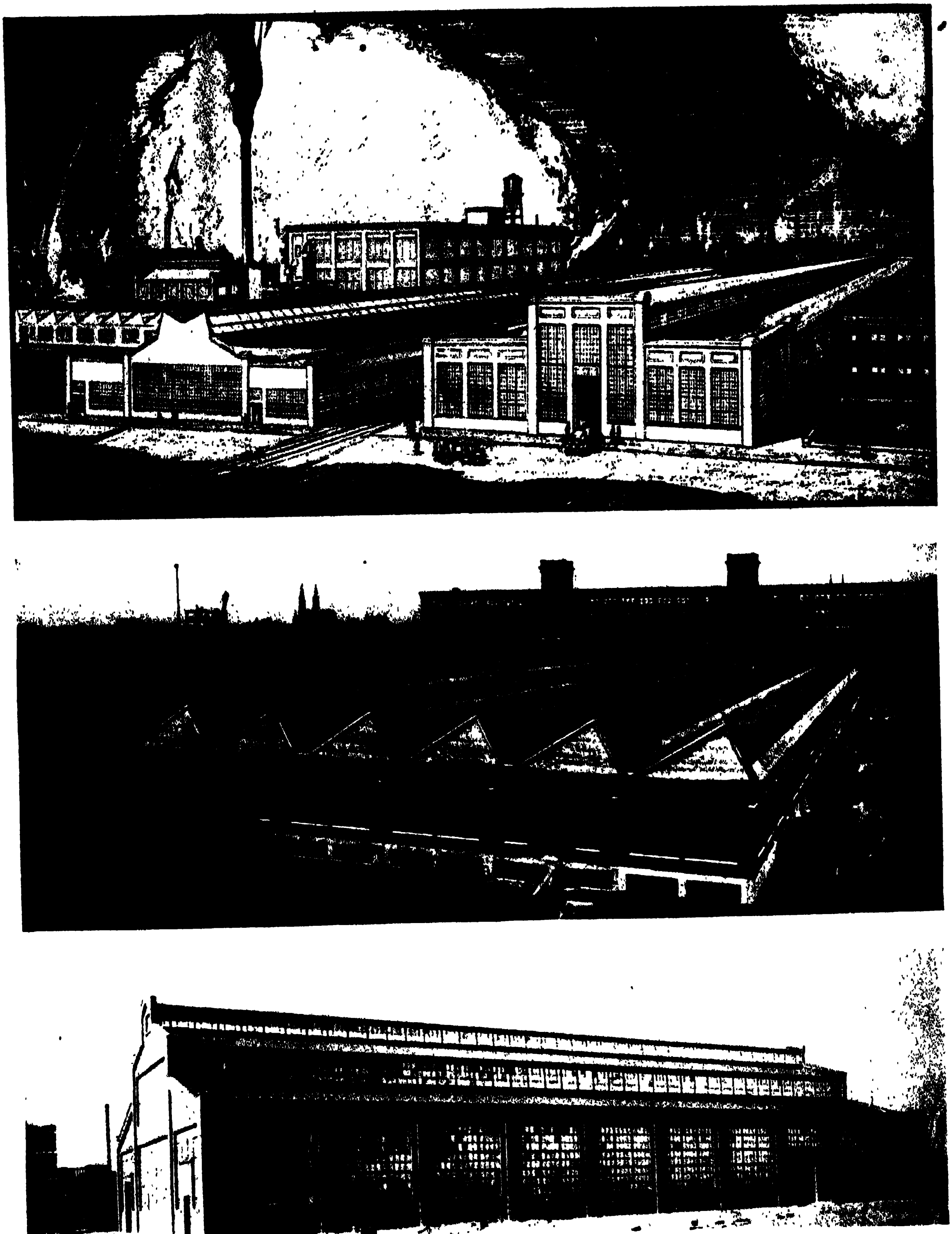


FIG. 559

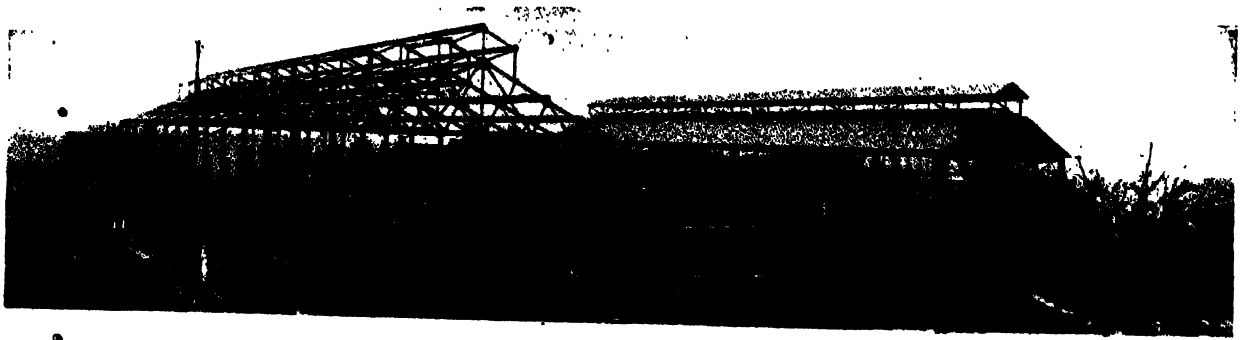
PART VI

MILL BUILDINGS



Courtesy of the Austin Company

PLATE 42 TYPICAL EXTERIOR VIEWS
MILL BUILDINGS



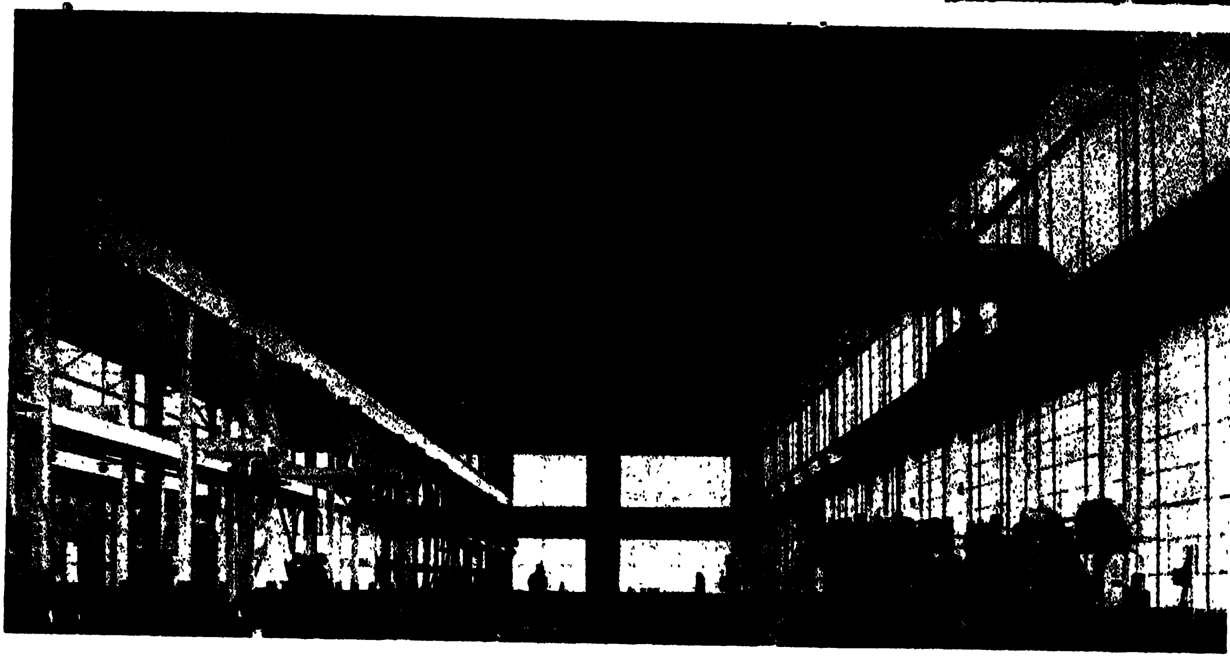


PLATE 43 TYPICAL INTERIOR VIEWS
MILL BUILDINGS

Courtesy of the Austin Company and the Cleveland Crane Company

CHAPTER 30

MILL BUILDINGS*

349. General Considerations.

A distinctive type of structure which is built for a definite use is the mill building. As the name implies, it is employed for mill work, or heavy manufacture, such as foundries, forge shops, rolling mills, machine shops, textile mills and the like, and includes a large variety of factory designs. In general, such a structure is not a manufacturing building in its broad sense, but is designed more as a part of the equipment of a definite process of manufacture. The architectural features are often secondary, although the appearance deserves consideration, and there is a tendency in modern practice to improve such buildings in this respect, as well as their surroundings. Figure 560 shows a perspective view of a typical building. The term "mill building" should not be confused with "mill construction" (Book 1).

Such structures must be carefully planned for the industrial work they house, so that production may be carried on at a minimum cost. Some of the important considerations are:

(1) The selection of the site,

(*a*) the area required for immediate use and that for future extensions.

(*b*) the location with respect to sources of power and raw materials, available labor, and facilities for shipping by rail or water.

(2) Height as opposed to area (Art. 350), which is largely influenced by the cost of land, and handling of materials.

(3) The machinery arrangement as affecting column locations, the styles of cranes, hoists, ducts, belt spaces, machinery foundations and the like.

(4) Heating, lighting, power, and ventilation.

* It is not the intention to give an exhaustive treatise on the subject of mill buildings here, but only to discuss the main general features, as there are several excellent textbooks on this subject. Two of these are: "The Design of Steel Mill Buildings and the Calculation of Stresses in Framed Structures," by M. S. Ketchum, — McGraw-Hill Book Co., Inc., and "A Treatise on the Design and Construction of Mill Buildings and Other Industrial Plants," by H. G. Tyrrell, — M. C. Clark Publishing Co.

350. Multi-story vs. Single Story Buildings.

The above considerations are usually studied by a mechanical engineer except possibly those of height compared with increased area. **The majority of mill buildings in modern practice are one story in height,** with as few dividing walls as possible. This feature is limited by building codes and fire insurance rates. Figure 561 shows a comparison of these types. The sizes and weights of the products to be manufactured, as well as the machinery to do the

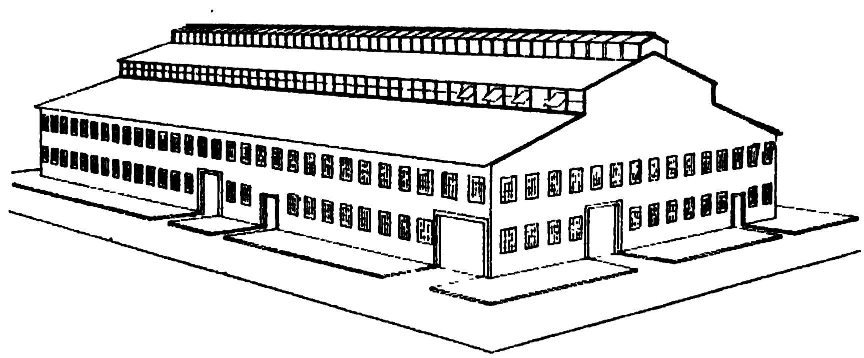


FIG. 560

work, are important considerations. The relative costs of the two types of buildings per square foot are also factors. The cost of a single story building of the same gross area as a multi-story structure is usually considerably less. Gross area, however, is not the real objective, and the maximum unobstructed **usable floor space** is the vital consideration.

Some of the advantages in favor of a single story building are:

(1) The operating costs are usually less. The labor charges are from 5 to 10% less because supervision is more direct. Fixed charges are smaller, as better ventilation, light, and heat may be obtained.

(2) The floor construction is cheaper as it is on the ground, and machinery foundations are more easily provided. Heavy live load requirements make superstructure floors expensive.

(3) The cost of handling materials is less, as

no elevator shafts and their enclosure walls are required. No stairways enclosed by fire walls, or fire escapes are necessary in a one-story building. Efficient routing systems may be employed, as the materials may be shipped from, or delivered to, any department with relative ease.

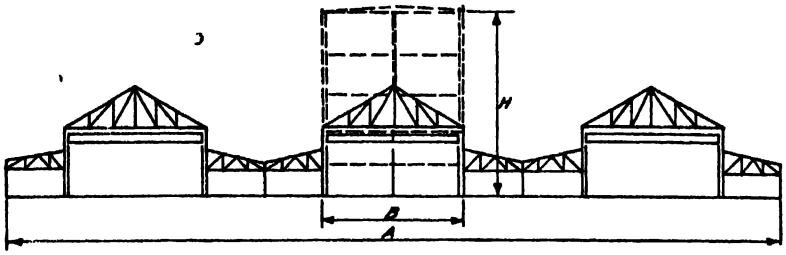


FIG. 561

(4) Future extension is possible in any direction on the lot with less expense. The time required for erection is also less.

(5) Less fire risk is usually involved.

The following* tabulation shows a comparison of the relative costs of the two distinct types, one of 6 stories, 60′-0″ × 200′-0″, the other a single story structure 150′-0″ × 480′-0″:

MULTI-STORY BUILDING

Stairs	2,160 sq. ft.
Elevators	1,512 sq. ft.
Approaches	3,600 sq. ft.
Outside Walls	3,120 sq. ft.
Columns	2,976 sq. ft.
Total	13,368 sq. ft.
Total Area	72,000 sq. ft.
Area Lost	13,368 sq. ft.
Usable Area	58,632 sq. ft.
Per Cent of Usable Area	82%
Land Required	½ acre

SINGLE STORY BUILDING

Outside Walls	1,260 sq. ft.
Columns	1,168 sq. ft.
Total	2,428 sq. ft.
Total Area	72,000 sq. ft.
Area Lost	2,428 sq. ft.
Usable Area	69,572 sq. ft.
Per Cent of Usable Area	96%
Land Required	2 acres

	MULTI-STORY	SINGLE STORY
Building Complete	$164,000.00	$122,000.00
Land, $6000 per acre	3,000.00	12,000.00
Total Price	167,000.00	134,000.00
Price per sq. ft. building, including heating, lighting, plumbing and elevators	2.27	1.70
Price per Usable sq. ft. of building	2.77	1.76
Price per Usable sq. ft., land and building	2.85	1.93

* Developed by the engineering department of the Austin Company, Engineers and Builders, Cleveland, Ohio.

"If a multi-story building of smaller dimensions and a greater number of stories had been used, a greater advantage in favor of the single story type would have resulted. Or if a single story building with longer spans had been selected an even greater advantage for this type would have been obtained. After several trials it was found that these two examples give a basis of comparison fair to both types.

"The specifications for both buildings include fire-resisting materials of standard quality which meet the requirements of present-day building codes. Similarly they meet requirements of insurance companies for reasonably low insurance rates. The same unit prices were used in both cases in so far as they applied. Heating, lighting and plumbing costs, as well as the cost of elevators, were included.

"It should be understood that either of these buildings could be embellished architecturally at increased cost. **The prices given here are submitted only as a basis for comparison, because variations in local conditions, as well as changing material prices and labor rates, prevent doing otherwise.**

"Notwithstanding the fact that Multi-story buildings are usually built on more expensive land than Single Story buildings, the same price for land has been used for this item in each case. This is shown in the charts."*

351. Types of Enclosures for Single Story Buildings.†

Since less fire risk is involved in a single story building than in a multi-story structure, the typical building of the former type is usually made with a structural steel frame, clothed with a covering. While unprotected steel is lacking in fire resistance, the cost to enclose it is higher than the usual investment will warrant. Precautionary measures should of course be taken. Some of the materials used in the walls and roof are fire-resistant, but the trusses and other framing are commonly exposed. Fire curtains, as shown in Fig. 562, are sometimes used to separate the sections of the building. These are made of asbestos, or sheathing of various materials. Wood framing and trusses have been used in the past, but steel is now more economical and desirable, generally. Concrete framing, especially in sawtooth types of buildings, is being used more lately. It offers better fire protection and begins to approach structural steel in cost, although it is not as adaptable to shock and impact stresses unless heavily massed.

The roof is generally formed with some light material such as corrugated sheeting (Arts. 164 and 358), asbestos protected metal (Art. 356), cement tiles, and so on. The walls may be any of three general types (although there are others), namely,

(1) Masonry bearing walls, upon which the roof trusses rest, strengthened by pilasters. The trusses are slightly lighter than in other types as they are subjected to less wind action.

† Enclosures for multi-story buildings are discussed in Parts II and III. The characteristics of design are similar to any building of this kind.

(2) Masonry curtain walls, which are commonly 8″ thick between the columns supporting the roof trusses. These are cheaper for wall construction than those in (1) but offer less resistance to vibration.

(3) A self-supporting frame with a light wall covering, such as corrugated sheeting or 2″ concrete and expanded metal. Ribbed lath is sometimes used with the latter to eliminate furring channels.

columns of light buildings are often made of a single angle, not less than a $6 \times 6 \times \frac{1}{2}$. The intermediate supports in the end frames are commonly I beam columns if the loads are light. The side columns are usually made of four angles laced. These are subjected to wind pressure and it is desirable to obtain as large a radius of gyration as possible in the direction of the bending, which is commonly perpendicular to the wall.

There are a number of names commonly applied

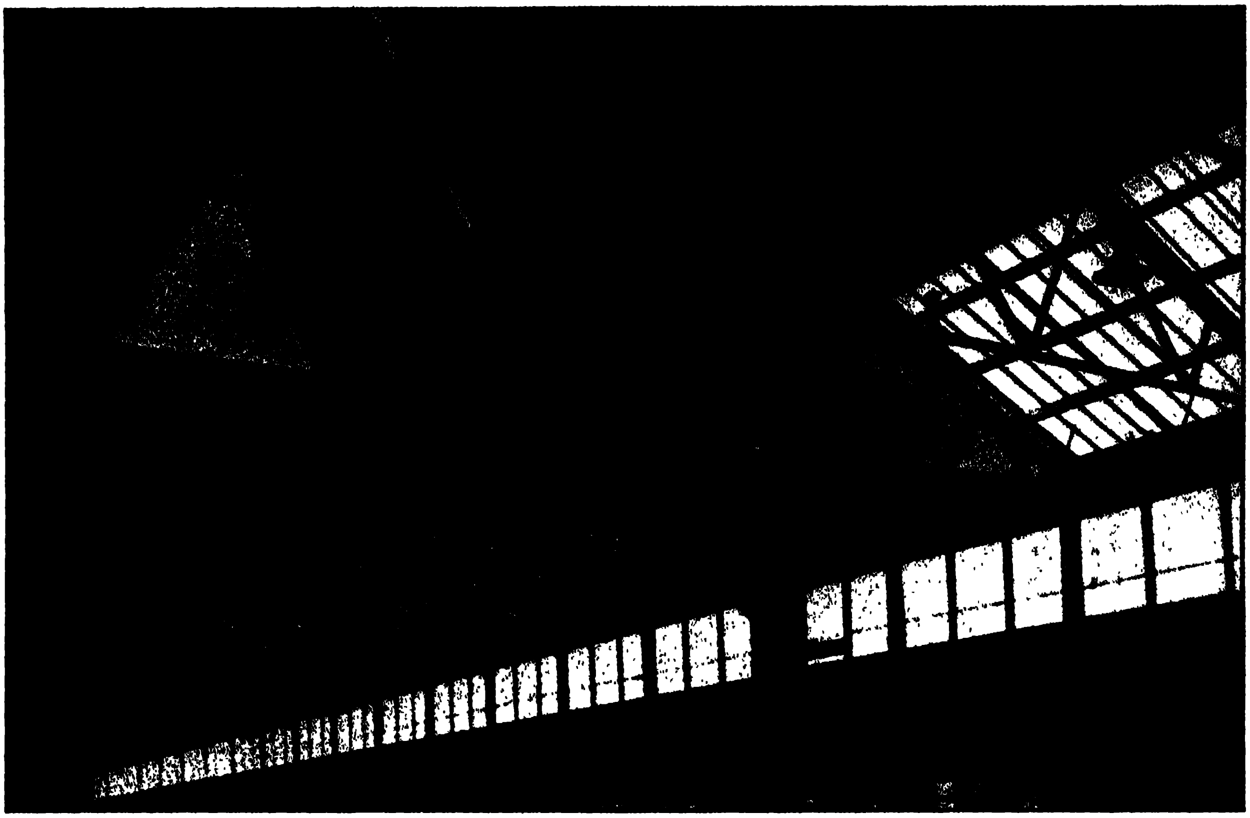


Fig. 562*

The relative values of ventilation, lighting, heating, and condensation must be considered in the selection of a type of wall. In many cases, however, the light covering (3) is used.

352. Features of a Typical Mill Building.

Figure 563 shows the important features of a single story mill building, in which the wall covering is of a light material. Each roof truss and the columns that support it are called a **transverse bent,** as illustrated in (*d*). A **bay** is defined as including all the construction between two bents. The construction corresponding to a bent at the ends of the building is called the **end frame.** The corner

* Courtesy of the Boston Manufacturers' Mutual Fire Insurance Company.

to the parts of the frame. In the end frame the member which conforms to the roof slope is called the **end rafter.** This is usually a channel section. The horizontal member at the low point of the roof is the **end strut.** The corresponding members in the side elevation are called the **eave struts.** These are commonly made of two angles starred or a pair of channels (Art. 363). The intermediate members which carry the siding are defined as **girts,** and are made of angles or light channels. These beams are attached to the columns, either to carry the covering directly or the sheathing upon which the siding is placed.

Many buildings have a **monitor** in the roof, in which louvres or adjustable sash are placed to provide ventilation and to remove obnoxious gases

which would otherwise collect under the roof.* The vertical height of the monitor is called the **clerestory.**

The trusses are usually stiffened by **kneebraces** at their juncture with the columns. The roofing is carried by the usual system of purlins (Art. 169), or for special types of roofing, by a series of sub-purlins. The typical building commonly has a set of **diagonal bracing rods** in the plane of the top chord, as in (*c*), a set of bracing angles in the plane of the bottom chord, as in (*b*), and diagonal rods in the ends and sides, as in (*e*) and (*a*). These serve to resist the action of the wind, and the vibration of machinery, and to aid in erection. **Sag rods** (suspenders) are often used to stiffen the purlins and girts against weight-deflection.

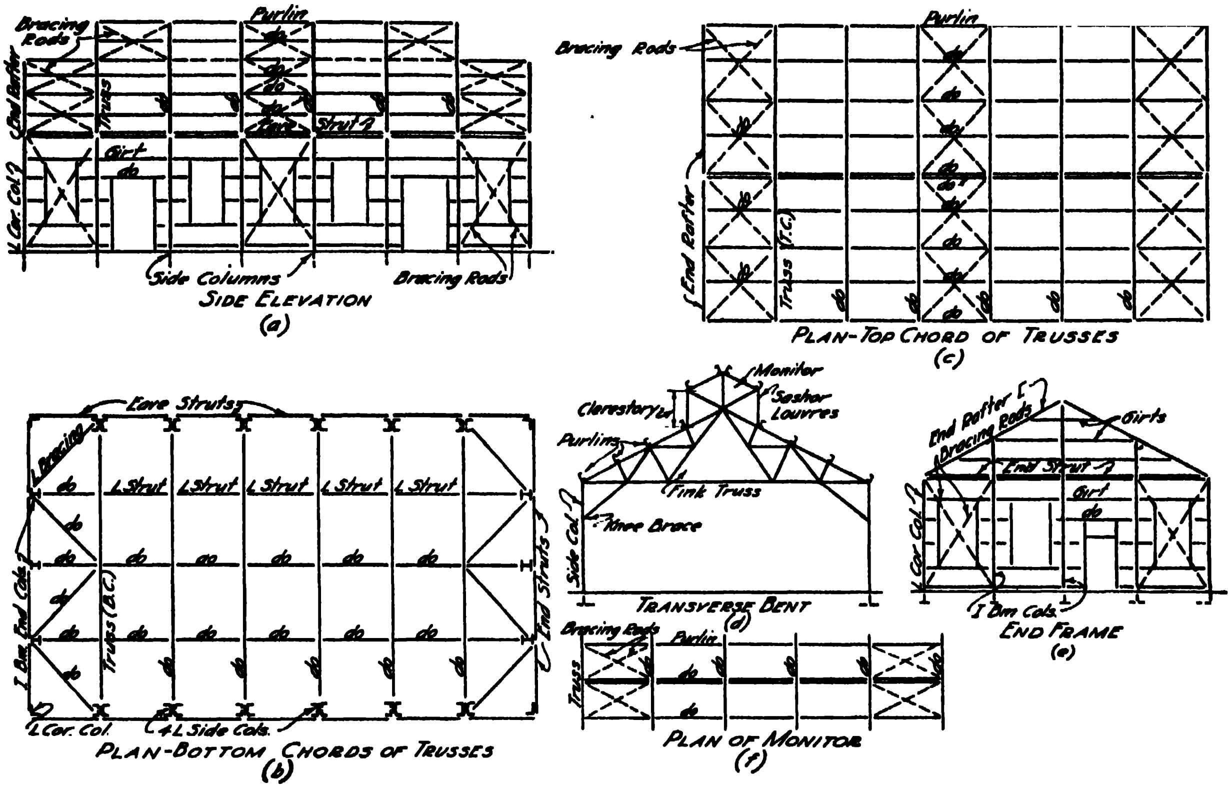


FIG. 563

There are a great number of types of transverse bents. Figure 564 shows the more common sections employed. **The Fink truss** (with a ¼ pitch) **is most commonly used** because of its many advantages (Art. 201). In some bents, **wing trusses** are used to provide side sheds or "lean-to's," as in (*g*), so that small operations may be carried on in these spaces. It is desirable to apply the loads from these side sheds as far down the main columns as possible. In some structures, **balcony levels** are provided for light manufacturing. The monitor frames may also be varied, as illustrated in Fig. 565, but they are most commonly made as in (*a*). The usual type is a longitudinal monitor, although cross monitors or box skylights are occasionally employed.

Cranes are used in many mill buildings. Inasmuch as the crane girders are generally supported by the columns and are not a part of the roof design, they and their columns are discussed later (Arts. 369–371). Skylights are sometimes introduced in the plane of the roof, as shown in Fig. 564 (*d*). They are generally used when the buildings are over 80′-0″ wide and vary in size from $\frac{1}{10}$ to ¼ of the floor area covered.

353. Ground Floors.

The type of floor which is placed on the soil to serve as a basement, or ground floor, depends upon the nature of the building. These may be classified as:

(1) rigid, such as brick or concrete,

(2) semi-elastic, such as mastic sheet asphalt, or wood blocks.

(3) "elastic," which are usually dirt floors.

For types of buildings in which the basement floor is used for storage or trucking, a concrete paving,

* If no smoke or obnoxious gases are to be encountered, a patented ventilator at the ridge may be used instead.

as in Fig. 566 (*a*), is most common. This is 4″ or 5″ thick and usually of 1 : 2½ : 5 concrete with the finish floated integrally. It may generally be placed directly upon the soil if the latter is dry, although a work is to be done, yellow pine plank floors may be used. When hot metal is to be handled, as in foundries and forge shops, a dirt floor is the common type. A floor of vitrified brick laid on a plank

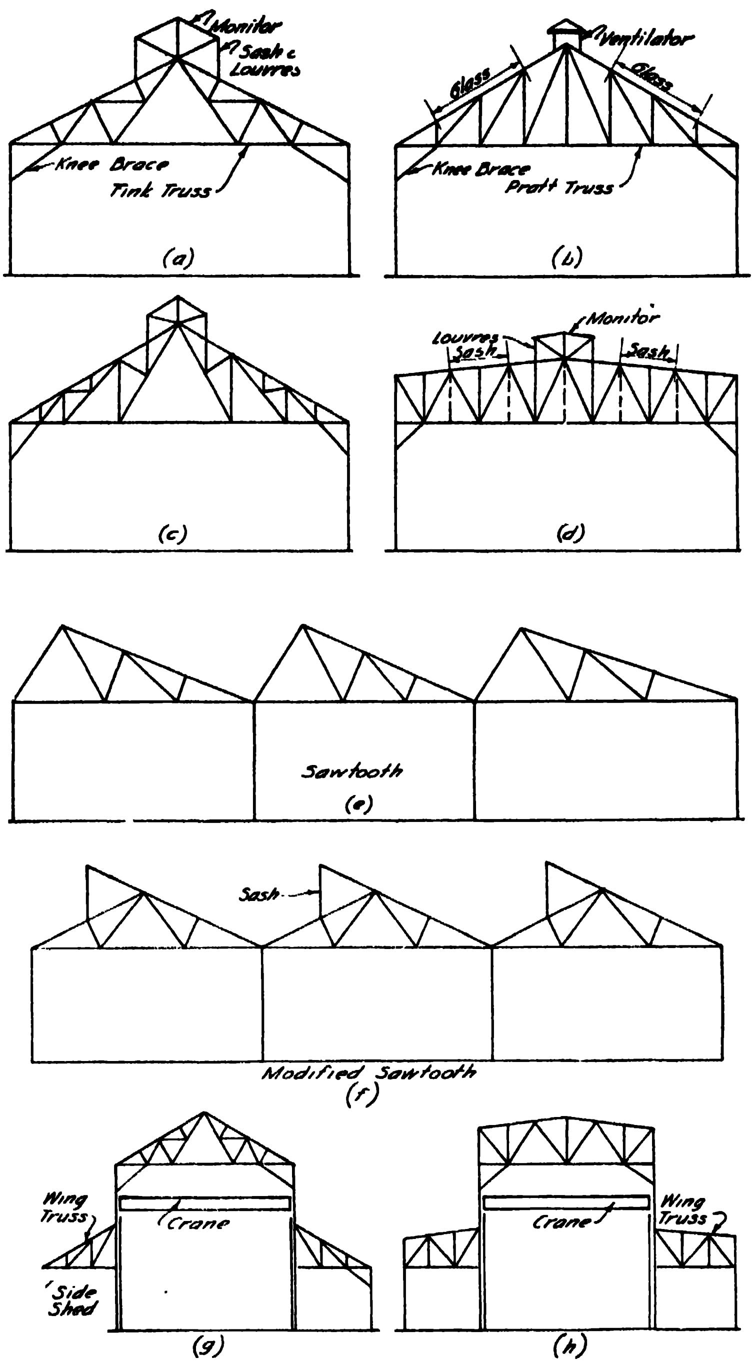


Fig. 564

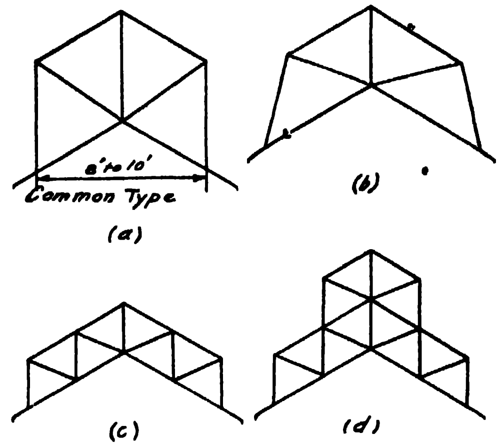


Fig. 565

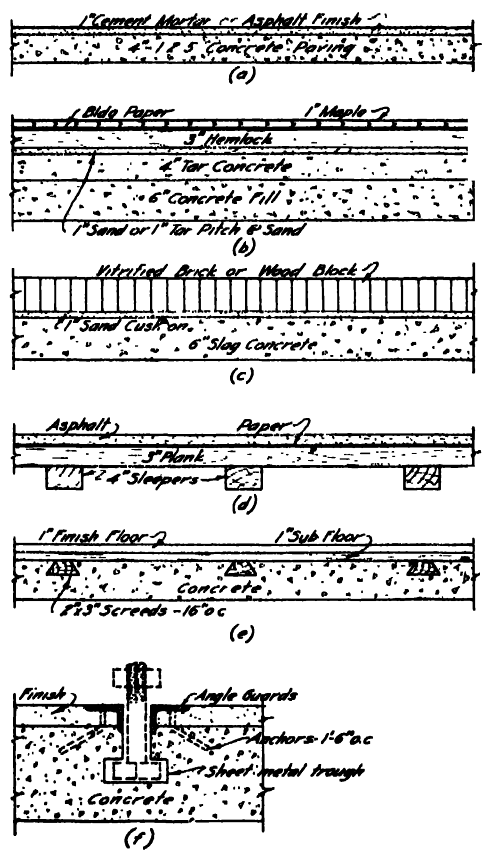


Fig. 566

cinder fill is sometimes used under the paving. For floors where there is to be considerable human occupancy a wood finish floor is best, although a mastic or wood block floor is good. Where rough foundation, waterproofed on the underside and imbedded in sand, may also be used. Various subfloors are employed, according to requirements, as indicated in Fig. 566.

354. Types of Roofing.

The most common type of roofing material which has been used in the past, particularly, for small mill buildings, is corrugated steel sheets (Art. 164). Other roofings have been more or less employed, according to the special requirements of a given project (Chap. 13). In addition to those previously discussed, comparatively new types of roofing have also come into use in recent practice. These have advantages especially adaptable to mill buildings, and for that reason, they are discussed in the following articles.

355. " Steel Deck " Roofs.

A recent type of roof supporting material is the " steel deck " roof.* It is particularly adaptable when the roof is not of complicated design, that is, not cut up by gables, dormers, gutters, and the like. Figure 567 shows the characteristics of the construction. In general, #18 gauge copper alloy steel plates are used, supported by a system of subpurlins and stiffening angles. The latter are firmly attached to the plates by patented bent locking plates. The size of the sub-purlins is varied according to the spans. Some of the advantages claimed are:

(1) Quick erection, as it may be covered immediately with any standard built-up roofing, regardless of weather conditions,

(2) There are no ridges, open spaces, bolts, rivets, or projections involved.

(3) It is light weight and non-combustible.

The weight averages 3.1#/□′ and the construction is designed to carry a live load of 40#/□′. Standard 8′-0″ lengths are furnished.

356. Asbestos Protected Metal.†

A form of roofing which is made of steel sheeting and usually corrugated, is asbestos protected metal. This material is also used for siding and the incident joint coverings. It is made rust and corrosion-proof by encasing both surfaces and edges of the sheets by three impervious protective coatings, namely,

(1) asphalt,
(2) non-perishable asbestos felt, and
(3) a heavy waterproofing envelope, the material of which is specially refined.

The four materials are bonded together by special machinery. The sizes of the sheets and spacing of the supporting purlins are similar to those of plain corrugated sheets (Art. 164).

357. Patented Slabs.

Besides the patented types of roof slabs which have already been discussed,‡ there are several other kinds upon the market. These in general are used in a manner similar to the ordinary concrete slabs, as far as the arrangement with respect to the purlins and other supports is concerned. The materials used for the slabs, however, are varied according to the patented combinations. Each has features which are claimed to be an advantage in one respect or another. Among such types of slabs are Porete, Structolite, and Flexnercrete.

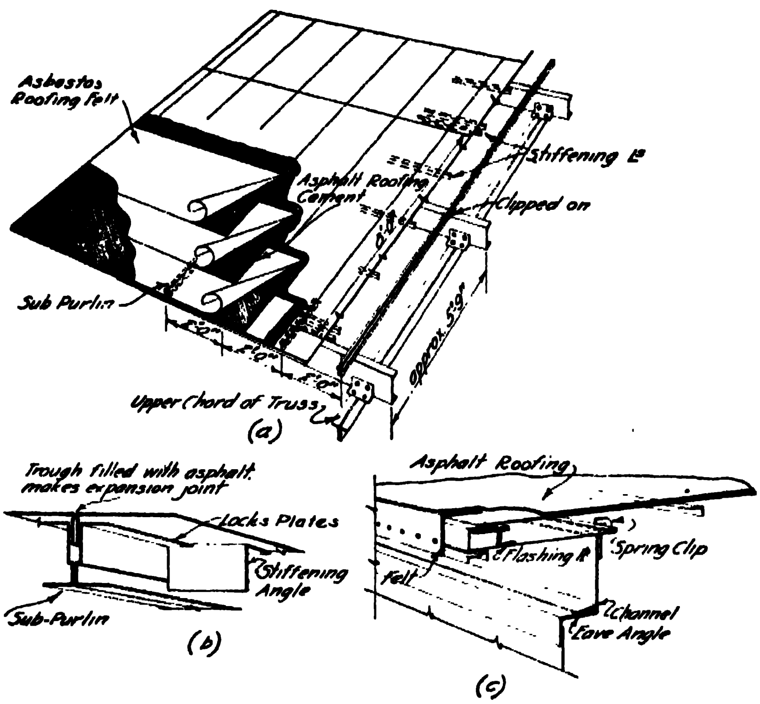


FIG. 567

Other patented preparations which are used in connection with providing a carrying surface for some types of roofing, such as slate, shingles, and certain types of tile, are on the market to replace the use of cinder concrete fill and screeds. These are also used for floor construction when a wood finished floor is desired. The materials are particularly useful for steep roof slopes, dormer window construction, and so on. Some of the common materials of this type are Nalecode, and Flexocrete.

* Manufactured by the Truscon Steel Co., Youngstown, Ohio.

† Manufactured by the H. H. Robertson Co., Building Products, Pittsburgh, Pa.

‡ Refer to Arts. 166 and 167.

358. Corrugated Siding.

The characteristics and typical features of corrugated sheets have been discussed in Art. 164 as applied to roofing. Similar details apply when this material is used for siding. It is sometimes referred to in the trade as "corrugated iron," but it is in reality steel. The #20 and #22 gauges are usually employed for siding with $2\frac{1}{2}''$, $2''$, or $1\frac{1}{4}''$ corrugations, the last being most common. Where warmth is particularly desired, the inside of the building is sometimes lined with #26 gauge, $1\frac{1}{4}''$ corrugated material. Cross pieces are usually placed in between the girts 2′-0″ to 3′-0″ o.c. Some engineers prefer to use wood blocks bolted to the girts and nail the sheeting to these. The material is fastened by nails in the troughs of each alternate corrugation, 2″ above the lower end of the sheet (1″ above top end of the under sheet). This allows a sheet to slide 1″ in 32″ in settlement before the nail strikes the upper end of the lower sheet. The laps on the sides and bottom are made one corrugation and the side laps should not be nailed. Care should be taken to have the projecting edges of the sheets at the eaves and gable ends of the roof securely fastened, otherwise the wind will loosen them. Figure 568 shows typical details for attaching the sheets to the girts (see also Fig. 259 for the attachment to the purlins). The material is often furnished as a sub-contract. Shop drawings must be made to show the sizes, locations and details for the sheets.

Standard pieces are manufactured which provide for gable cornices, ridge rolls (to cover the joints at the ridge), hip and valley flashings, corner capping, chimney and wall flashings, door and window casings, roof and valley gutters and downspouts, ventilators, and so on (see M. S. Ketchum's Structural Engineers' Handbook *).

If no sheathing or paper is placed under the purlins in a roof covered with corrugated sheeting, moisture will collect, due to the difference in temperatures outside and inside the roof. For this reason, anti-condensation lining is often specified. This is fastened to one eave purlin, over the ridge, and down to the other eave. It consists of a series of layers. The first is a #19 gauge galvanized wire netting of 2″ mesh (chicken wire) placed transversely to the purlins, with the edges $1\frac{1}{2}''$ apart and laced together with #20 brass wire. When the purlins are more than 4′-0″ o.c., a #9 galvanized wire is stretched in between, to hold up the netting. The second layer consists of asbestos paper weighing about 14#/100□′, which is about $\frac{1}{16}''$ thick. All holes are patched with 6# paper. This covering is lapped 3″ and joints broken every 12″. Two layers of tar building paper are then placed over the asbestos paper. Sometimes saturated felt is used instead. The corrugated sheets are then laid. Tin washers $1'' \times \frac{1}{8}'' \times 4''$ and stove bolts are used to fasten the sheets where there is danger of tearing the lining.

Prob. 358a. Find the total allowable uniform load for $2\frac{1}{2}''$ corrugated sheets, #20 gauge, if the purlins are 8 ⊏ $11\frac{1}{4}$ spaced 4′-0″ back to back (see Art. 164). What load is this in #/□′?

Prob. 358b. What gauge corrugated sheet should be used for a load of 30#/□′ if the purlins are 7 ⊏ $9\frac{3}{4}$, spaced 4′-6″ back to back? Use $2\frac{1}{2}''$ corrugated sheets.

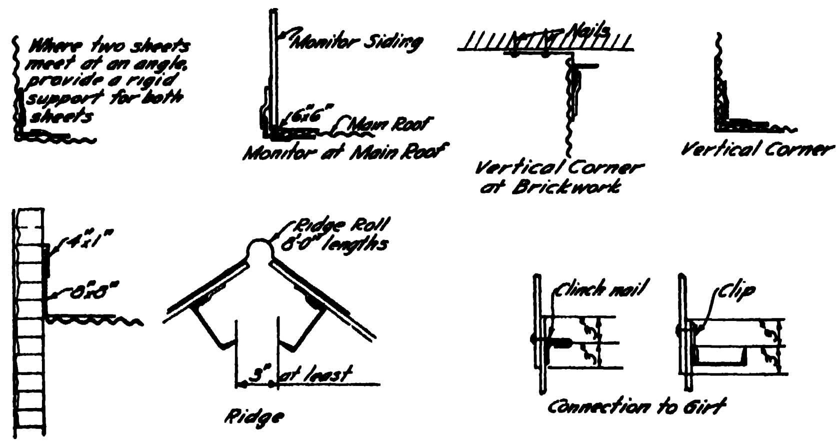


FIG. 568

359. Design of Purlins.

The typical design of purlins has already been discussed in Art. 169. Tie-rods are often used to stiffen the purlins, as indicated in Fig. 268 and induce a variation in their design. These are called "sag rods" also, as they prevent the tendency of the purlins to sag toward the eaves at their centers. They should extend in a line from eave to eave over the ridge. The design of the tie-rods is discussed in Art. 170.

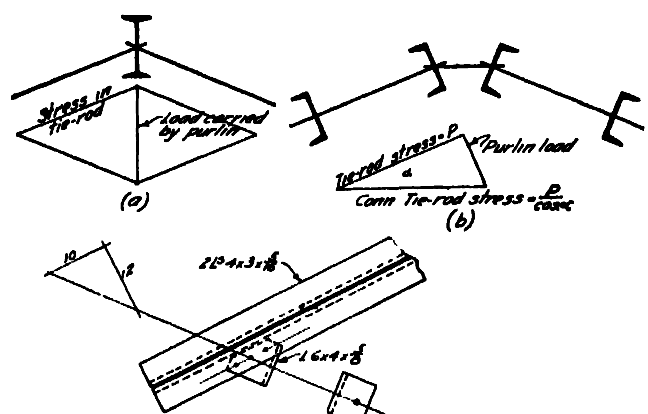


FIG. 569

* McGraw-Hill Book Co., Inc.

These are usually $\frac{5}{8}''\ \phi$ or $\frac{3}{4}''\ \phi$, $\frac{5}{8}''$ being the minimum, while $\frac{3}{4}''$ is generally used to maintain common punching of the rolled sections. Rods are preferred to bars as they supply the minimum surface to corrosion for a given cross-section.

The stress in a tie-rod may be graphically represented as in Fig. 569 (*a*). That in a connecting rod at the ridge is shown in (*b*). Tie rods are usually attached by employing beveled washers, although angle clips may be used, as illustrated in Fig. 569 (*c*).* Minor rods are often bent in order that flat washers may be used. One rod is commonly used at the center-line of the bay. If a selected size of rod figures out as overstressed, two lines of rods should be employed, one at each third-point of the purlin span, or a larger tie-rod should be used. Some specifications limit the spacing to 30 times the flange width of the purlin. (See Art. 11.)

Illustrative Prob. 359a. If the slant length of a roof is 36′-0″, and the trusses are 16′-0″ o.c., and the maximum wind load component parallel to the roof is 10.2#/□′, what size of tie rod should be used?

Assume line of rods at center-line of panel
Tributary area $= 36 \times \frac{16}{2} = 288□'$
Force $= 288 \times 10.2 = 2940\#$

$$\text{Area required} = \frac{2940}{16{,}000} = 0.184□''$$

Use $\frac{5}{8}''\ \phi$ Tie-rods.

Prob. 359b. If the span of a truss is 60′-0″, the inclination 30° with the horizontal, the maximum wind load against a vertical plane 30#/□′, and the trusses are 14′-0″ o.c., what size of tie rods should be theoretically used?

360. Design of Girts.

The girts must support the weight of the siding materials which they carry, as well as their own weight, and they must resist the wind pressure against the side of the building. Usually, the effect of the weight of the siding is small compared with the stresses resulting from wind pressure. The weight of the girts varies from $1\frac{1}{2}\#$ to 3#/□′ of the net surface carried. The weight of corrugated sheeting is from 1# to 4#/□′ according to the gauge of the metal. The girts should be capable of safely resisting 10# to 20#/□′ of wind pressure, depending upon the exposure.

The spacing of the girts depends to some extent upon the arrangement of the doors and windows. They should be planned so that certain girts will coincide with the heads of doors and windows and with the sills of windows. The spacing also depends upon the safe limits of the corrugated sheets if the latter are used (Art. 358). The following represents average spacings for girts for ordinary loads:

Gauge of Sheets	Walls	Roofs
#26	3′-6″	2′-6″
24	4′-0″	3′-0″
22	4′-6″	4′-0″
20	5′-0″	4′-6″
18	6′-0″	5′-0″
16	7′-0″	5′-6″

It is preferable to have the sheets span over two purlin spacings. Since the maximum length of the sheets is 10′-0″, a lap of 6″ allows a **usual maximum spacing of 4′-9″**. From a study of the previous safe spacings, it may be seen that there are certain gauges which are most economical to use. **The #22 and #24 gauge sheets are commonly used** for siding, and the #18 and #20 gauges for roofing.

The sizes of the girts are usually proportioned by experience and practical considerations and not by calculations. The following represents typical minimum sizes for varying conditions:

Bays	Angles	Channels
10′-0″ & 12′-0″	$3 \times 2\frac{1}{2} \times \frac{1}{4}$	$4 \sqcup 5\frac{1}{4}$
14′-0″ & 16′-0″	$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$	$5 \sqcup 6\frac{1}{2}$
18′-0″	$4 \times 3 \times \frac{5}{16}$	$6 \sqcup 8$

Calculated sizes may in some cases be smaller, but the above minimums should be used in all cases. For extreme conditions, larger calculated sizes may result, but the above sizes have been safely used on many jobs. It is seldom that a full wind pressure is developed.

Illustrative Prob. 360a. Calculate the required size of channel girt for a 14′-0″ bay if the spacing of girts is 3′-6″. Design for a wind pressure of 20#/□′.

Load per linear ft. $= 3.5 \times 20 = 70\#$
Assume effective span $= 14'\text{-}0'' - (1'\text{-}0'') = 13'\text{-}0''$
$M = 1.5\, w \cdot L^2 = 1 \cdot 5 \times 70 \times (13)^2 = 17{,}700''\#$

$$\frac{I}{c} = \frac{17{,}700}{16{,}000} = 1.1''^3 \text{ about 1-1 axis}$$

$3 \sqcup 4.0$ theoretically required.
Use $5 \sqcup 6\frac{1}{2}$ for practical reasons.

Vertical load per foot $= 3.5 \times 4 = 14\#$
$M = 1.5 \times 14 \times (13)^2 = 3140''\#$

$$\frac{I}{c}\,(1\text{-}1) \text{ for } 5 \sqcup 6\tfrac{1}{2} = 3.0''^3 \qquad \frac{I}{c}\,(2\text{-}2) = 0.38''^3$$

$$\text{Stress (2-2 axis)} = \frac{3140}{0.38} = 8{,}260\#/□''$$

$$\text{Stress (1-1 axis)} = \frac{17{,}700}{3.0} = 5{,}900\#/□''$$

$$\text{Total} = 14{,}160\#/□'' \quad \text{O.K.}$$

Sag rods (suspenders) are used to stiffen the channels in their weaker direction at their mid-points for spans 14′-0″ or more. These are $\frac{3}{8}''$ or $\frac{1}{2}''\ \phi$ rods staggered back and forth 3″, to make a tie to the eave or end struts.

* The use of clevises for such work has gone out of practice.

Figure 570 shows some common details for girt connections to the columns.

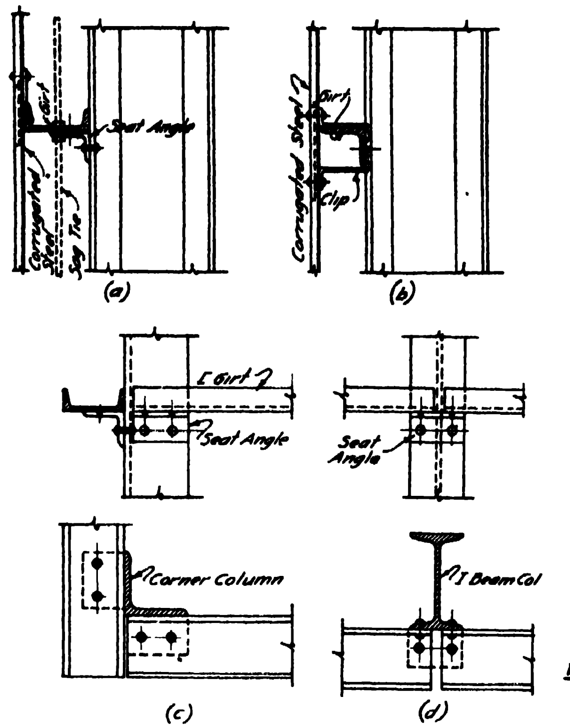


FIG. 570

Prob. 360b. If the spacing of girts is 3′-0″ vertically and their span 13′-6″, what size is theoretically required for a wind pressure of 25#/□′ for (a) channel girts (b) angle girts?

361. Types of Bracing.*

The effects of the wind on a mill building are resisted by a series of members, namely,

(1) the end struts,
(2) the lateral rods in the end frame,
(3) the eave struts,
(4) the lateral rods in the side framing,
(5) the top chord bracing,
(6) the bottom chord bracing, and the angle struts between the trusses in the plane of the bottom chords,
(7) the bracing between the monitor frames, and
(8) the knee braces in each bent.

The purlins in the roof become a part of the top chord bracing system. Some stiffness is also supplied by the intermediate girts in the side and end frames, but the amount is indeterminate, and usually its value is neglected in investigations, especially when a light covering for the walls is used. Their stiffness may be counted upon as so much additional protection.

The knee braces are used primarily to brace the bents against the wind pressure exerted against the sides of the building. Since they are attached to the bottom chords of the trusses at their upper ends, and to the columns at the feet of the knee braces, they induce stresses in the trusses and bending in the columns. This action requires a separate investigation and will be discussed later.

If brick walls are used instead, no permanent bracing in the side and end walls is necessary. Temporary bracing, however, should be employed, to aid in the erection and to resist wind stresses caused by pressure on the frame before the brick is laid.

362. Stresses in the End Frame.

The end frame resists the wind pressure on the side of the building for a length of one-half a bay.

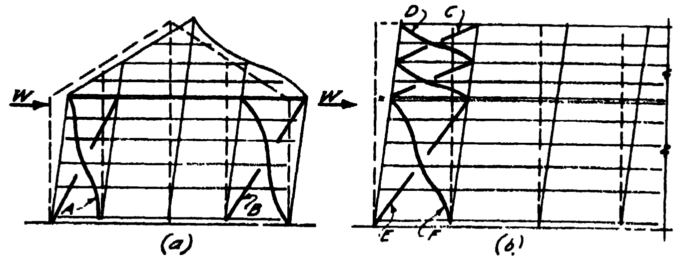


FIG. 571

Figure 571 shows an exaggerated case of the tendency towards failure in the side and end framing. This is all that must be provided for, since each interior bent is designed to resist the pressure on a length of one bay. A portion of the pressure is transmitted to the foot of each column in the end frame by virtue of the strength of the latter. It is reasonable to assume that the pressure for a height one-half way to the eaves is so disposed of. The **compression in the end struts** is then the pressure on the remaining tributary area. Thus for the following data:

Length of bays 16′-0″, height to eaves 20′-0″, rise of roof 15′-0″, wind pressure 20#/□′,

the area tributary to the end struts is $(\frac{20}{2} + 15)\frac{16}{2} = 200$□′. The compression is then $200 \times 20 = 4000$#. The selection of the sizes for the member then becomes that of designing a strut to resist this load (Art. 206). The length is the distance between the columns in the end frame (usually one-quarter of the width of the building). Ratio of slenderness is often a controlling feature. Two angles starred, or a pair of channels, as illustrated in Fig. 572, are

* For purposes of design and estimating, bracing in the side and end frames, as well as in the planes of the chords, averages 1#/□′ in weight.

commonly used. For light buildings, 2 ∟ 3½ × 2½ × ¼, or 2-6 [8.2, are often sufficient. These members must be strong enough to resist the local wind pressure exerted against them and to carry their proportion of the siding, the same as any of the intermediate girts (Art. 360).

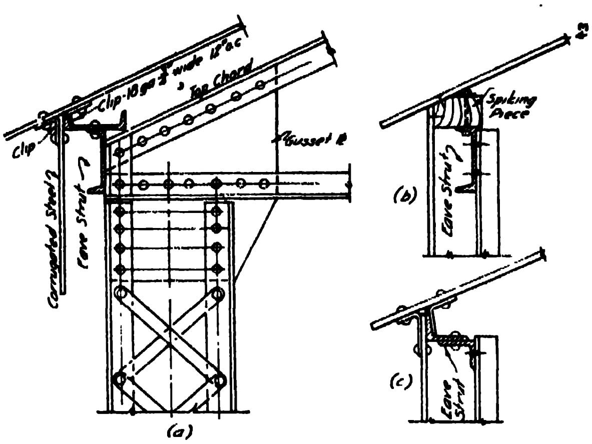


FIG. 572

Two bays of diagonal bracing are usually sufficient in the end frame, as shown in Fig. 563. They must resist the horizontal effect of the wind on the half bay of the side elevation, as previously discussed, or in other words, offset the compression in the end strut. This means that the diagonals must

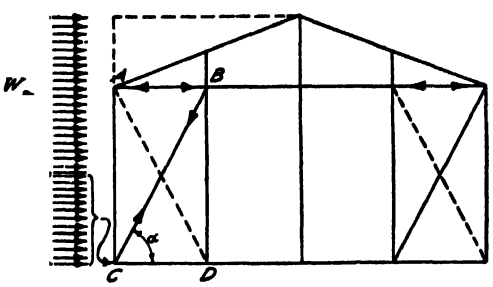


FIG. 573

supply tensile resistance. Since two sets of diagonals are to be used, each set is designed to resist in tension, one-half of the compression in the end strut. If the wind is blowing from the left, as indicated in Fig. 573, only the diagonal BC in the panel AB is in tension. If α is the angle between the diagonal and the horizontal, then the tension in BC is

$$T = \frac{\frac{1}{2}\text{ compression in end strut}}{\cos\alpha}. \qquad (S\text{–}91)$$

Single angles are used for these members in the better classes of work, although rods may be used for the sake of economy. The size must be sufficient to provide enough net section at the allowable stress (Art. 205). Usually 2½ × 2 × ¼ angles or ¾″ φ rods are sufficient and such sizes should be used as a minimum. Care must be used in planning the details so that a minimum interference with the windows is obtained.

The portion of wind load tributary to the bases of the columns for the data previously discussed is

$$\tfrac{1}{2}\text{ (height to eaves)} \times \tfrac{1}{2}\text{ (width of bay)} \times \text{pressure, or } \tfrac{20}{2} \times \tfrac{16}{2} \times 20 = 1600\#.$$

The forces on other columns may be calculated in a similar manner. These may be readily resisted by the stiffness of the base connections and anchorage, if properly designed for other requirements. The diagonal bracing rods produce some bending in the columns at their bases unless some horizontal resistance is provided. A **bottom strut,** similar to an end or eave strut, is sometimes used in large mill buildings, but in usual cases, the horizontal components of the stresses in the diagonal rods are small, and typical girts, placed at or near the points of the rod connections, are sufficient for this purpose.

363. Stresses in the Side Framing.

The **compression in the eave struts** is developed by the wind pressure acting upon a portion of the end of the building. As before, the pressure on the area up to one-half the height to the end struts is transferred to the bases of the columns by the strength of the latter. The pressure on the remaining area is brought to the eave by the end rafter and by the girts partially, but principally by the top and bottom chord bracing. In Fig. 574, the tributary area is

$$30 \times 10 + \frac{30 \times 15}{2} = 525\square'.$$

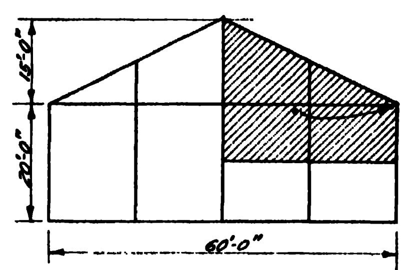


FIG. 574

If 20#/□′ wind pressure is used, the compression in the eave struts is 525 × 20 = 10,500#. The eave struts are commonly made of the same section as the end struts (Art. 362).

The **diagonal bracing** in the side framing must supply tensile resistance to oppose the compression in the eave struts, in a manner similar to that in the end frame. Bracing in the two end bays is usually sufficient for buildings up to 120′-0″ in length. If

the locations of entrance doors so require, it may be placed in the bays adjacent to the end ones. For long buildings, bracing should be placed in every fifth or sixth bay. Some stiffness is supplied by the bracing between the top chords of the roof trusses. This is not sufficient in itself and its chief value is in the erection of the frame. The amount of stiffness of the top chord bracing, in aiding the side framing, is indeterminate, and the usual procedure is to neglect its value and to develop the wind pressure by the lateral rods. Common practice is to assume that only two bays of diagonals resist the pressure. This is true for short buildings in which only two bays are used, and in long buildings the braced bays at each end are too far apart, so that only two bays should be considered as effective. Of course only one diagonal in a given panel offers tensile resistance at a time, depending upon the direction in which the wind is blowing. If P is the compressive force in the eave struts, and β is the angle between the diagonals and the horizontal, the tensile stress in one diagonal, based upon the above assumption, is

$$T = \frac{1}{2}\,\frac{\text{compression in eave strut}}{\cos\beta}. \qquad (S\text{-}92)$$

The same sizes of bracing material are generally used as in the end frames (Art. 362).

364. Top Chord Bracing.

The bracing in the planes of the top chords is used principally as an aid in erection and to relieve the roofing material of excessive strain. Diagonals are introduced in the end and other braced bays, as shown in Fig. 563 (c). With the wind blowing on one end of the building, only one set of diagonals is in tension. If these are considered only, an imaginary Pratt " truss " is formed, as illustrated in Fig. 575. The top chord of the first roof truss forms one chord of the bracing truss, and the end rafters form the other chord of the bracing truss. The purlins at the panel points form the web compression members, and the diagonal bracing rods the web tension members. This " truss " is in reality in two planes because of the two roof slopes. The panel point loads on this " truss " may be found by referring to the end frame, calculating the tributary areas, and multiplying these by the specified intensity of wind pressure. The tributary areas are illustrated in Fig. 575. The stresses may then be determined in the usual way, by calculation (Art. 190), or by a graphical stress diagram (Art. 189). The maximum tension in the diagonals is thus obtained and the size of rods determined. In mill buildings of average size, $\frac{3}{4}''$ ϕ or $\frac{7}{8}''$ ϕ rods are usually sufficient. The size is of course kept constant for all diagonals for simplicity of details.

365. Bottom Chord Bracing.

The bracing in the plane of the bottom chords of the roof trusses also becomes an imaginary " truss," as shown in Fig. 575. The bottom chord of the first

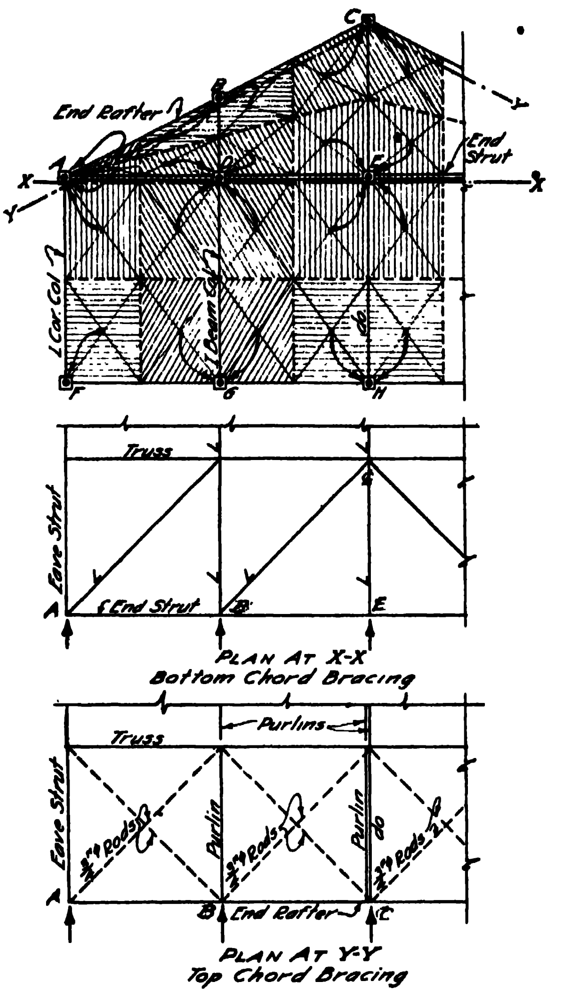


Fig. 575

roof truss forms one chord of the bracing truss, and the end struts form the other chord. A series of " web members " are supplied by the bracing angles. These prevent undue deflection of the columns in the end frame and of course stiffen the bottom chords of the roof trusses. The stresses may be obtained as before by calculating the tributary areas of wind pressure on the end frame, determining the " panel loads," and proceeding as in any truss solution. Instead of using this procedure, some engineers proportion these angles by "experience." Minimum sized angles of $2\frac{1}{2} \times 2 \times \frac{1}{4}$ should be used in any instance and these may be increased according to the lengths of the bays.

Angle struts are usually employed in the plane of the bottom chords of the roof trusses along the length of the building between the panels of bracing,

as shown in Fig. 563. These do not receive stress, theoretically, but aid in stiffening the frame in a practical way. Single angles of a nominal size, often $2\frac{1}{2} \times 2 \times \frac{1}{4}$ minimum, are used, but a limiting ratio of slenderness will often govern. They should be sufficient in sections so that their deflections under their own dead weight do not exceed $\frac{1}{360}$ of their span lengths. The connections should be designed to develop the net sections of the angles by rivets.

Bracing in the monitor frame is often omitted because the areas subjected to wind pressure are relatively small. When used, it is principally an aid in erection. A **ridge strut** is sometimes used between the monitor trusses to stiffen the frame (Art. 212).

366. Stresses in a Transverse Bent.

The dead load and snow load stresses in the roof trusses of a transverse bent may be obtained by the usual methods, either graphical (Art. 189), or analytical (Art. 191). There are no stresses caused in the knee braces by dead and snow loads, except those due to the deflection of the truss. The latter are very small, and only involved investigations will determine their approximate values, and consequently they are neglected in ordinary practice.

For average mill buildings, some designers determine the wind stresses with the wind loads normal to the roof in the customary way (Art. 161) and obtain the maximum stresses in the roof truss accordingly. They arbitrarily make the knee brace a pair of angles (back to back) of the same size as those in the bottom chord of the truss. The effect

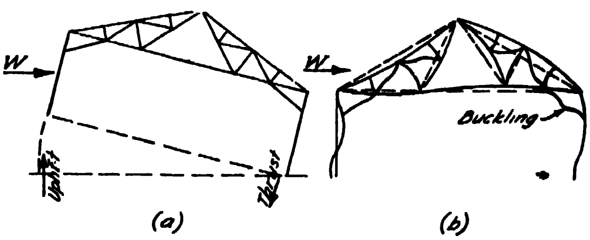


FIG. 576

of the stresses in the knee braces upon the stresses in the members of the truss proper is then neglected. Larger sized angles in the truss are seldom required for the extra stresses induced by the knee braces. The effect of the wind pressure against the side of the building upon the columns cannot be as simply neglected unless their size is arbitrarily made larger. The stress in the knee brace also produces some bending in the column. Knee braces may be omitted when the truss is attached to the columns at two points (Fig. 334), as these connections in themselves are sufficient to stiffen the frame against the action of the wind.

Many designers prefer to make a more careful analysis of the stresses in a transverse bent, however. Figure 576 shows the effect of the wind on a bent, that in (*a*) illustrating overturning, and that in (*b*), buckling of the leeward knee brace, inducing failure in the roof truss. Overturning is a remote cause of failure, but the tendency must be considered. Before any analysis can be made, it is necessary to assume a condition as to the degree of fixity at the bases of the columns and at their tops, and as to the type of knee brace connections. There are a number of assumptions possible, the more common of which are:

(1) columns hinged at the top, and fixed at the bottom,
(2) hinged at the top and at the bottom, and
(3) fixed at the top and at the bottom.

For many practical reasons, the usual assumption made is the last named above, (3). Riveted connections fix the tops of the columns to some extent. The degree of fixity developed is not a positive amount, and depends upon the details used, but it is definite enough so that pin ends could not be a reasonable assumption. The bottoms of the columns are more or less fixed by heavy bases and anchorage to the footings, as well as by the dead loads on the columns, and a "free end" condition is not a probable assumption. Other factors which complicate any theory are the stiffness of the knee brace connections, that of the roof framing and covering, and the variations between theoretical and actual wind pressures on large vertical and inclined surfaces (Art. 160).

One of the effects of the wind is to produce **direct stresses in the columns.** Tension is induced in the windward and compression in the leeward column. The first instance does not affect the design, as the tension tends to neutralize some of the compression due to dead load and would not ordinarily be large enough to exceed it. Obviously, the compression developed in the leeward column by the wind action must be added to that due to the vertical loads, to obtain the maximum value. Figure 577 shows the wind acting horizontally on all surfaces. Other theories assume the wind to act perpendicularly to all surfaces, but the former assumption is more common. The analysis of the effects of the wind is similar to that for portals in bridge design.

Let W = the total wind pressure, acting horizontally on the side of the building over one bay in length,
H_1 and H_2 = the horizontal reactions at the bases of the two respective columns,
V_1 = the vertical reaction, acting downward, on the windward column,
V_2 = the vertical reaction, acting upward, on the leeward column,
L = the distance between the center-lines of the columns, and
H = the total height of the building.

One-half of the horizontal pressure, W, is resisted at each base, or

$$H_1 = H_2 = \frac{W}{2} \qquad (\Sigma H = 0).$$

These values represent the tendency to shear off the anchor bolts of the columns, or to slide the bent sidewise. The resultant wind pressure is assumed to act at a point half-way up the height of the building, $\frac{H}{2}$, assuming the base of the column as the bottom of the building. Taking moments about B,

$$V_2 \cdot L = W \cdot \frac{H}{2} \qquad (\Sigma M = 0), \quad \text{or}$$

$$V_2 = \frac{W \cdot H}{2\,L}$$

$$V_1 = V_2 \qquad (\Sigma V = 0)$$

The above value is the direct compression induced in the columns by the wind. The tension may be thought of as the uplift the anchor bolts must resist.

The knee brace tends to produce **bending in the column.*** The maximum moment occurs at the foot of the knee brace, and the worse condition is in the leeward column. If fixed

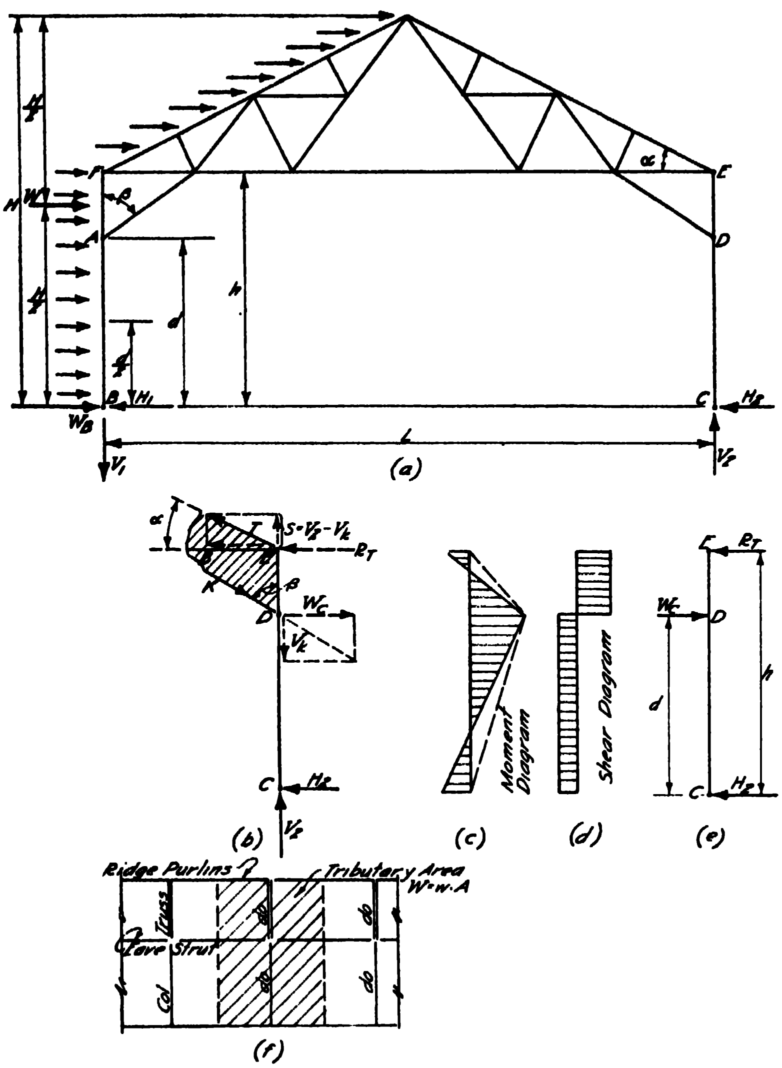


FIG. 577

ends, as previously discussed, are assumed for the column, the point of inflection is located at a point half-way up the height of the column from the base to the knee brace. The force, H_1, may be assumed as acting at this point. The bending moment in the windward column is then $(H_1 - W_B)d$, and in the leeward column is $H_2 \cdot d$, when W_B is the portion of wind load carried at the foot of the windward column. Since $H_1 = H_2$, the latter condition of bending is greater, so that it will usually control the design. The column may be considered as a beam fixed at the ends and supporting a concentrated force at its point of maximum moment. Figure 577 (e) shows the loading, and (d) and (c) the corresponding shear and moment diagrams. The first step is to calculate R_T and W_C. It is known that $W_C = R_T + H_2$. Taking moments about D in (e),

$$R_T(h - d) = H_2 \cdot d, \text{ or}$$

$$R_T = \frac{H_2 \cdot d}{(h - d)}. \qquad (S\text{-}93)$$

When R_T is known, $W_C = H_2 + R_T$. The maximum bending moment is then

$$\frac{W_C \cdot h}{4} \times \frac{8}{12} \text{ (when reduced for continuity),}$$

$$\text{or} \qquad \frac{W_C \cdot h}{6}.$$

When W_C is known, **the stress in the knee brace** may be calculated from

$$\text{stress } K = \frac{W_C}{\sin \beta} \text{ (Fig. 577 (b)).} \qquad (S\text{-}94)$$

A pair of angles may then be proportioned for this stress by the usual methods (Art. 206). As previously stated, these angles are commonly made the same size as those in the truss web member opposite the knee brace, unless larger ones are necessary.

The action of the knee brace also causes **induced stresses in the roof truss.** Referring further to Fig. 577 (b), the horizontal and vertical components of the stress K are W_C and V_K respectively. The net vertical effect at the point E is then equal to $S = V_2 - V_k$. The value of V_k is $W_C \div \cos \beta$. From the value of the vertical force at point E, the stress induced in the top chord, T, may be obtained, or

$$T = \frac{S}{\sin \alpha}. \qquad (S\text{-}95)$$

When T is established, the stress induced in the bottom chord of the roof truss, B, is

$$B = R_T - T \cdot \cos \alpha, \qquad (S\text{-}96)$$

in which α is the inclination of the top chord with the horizontal (R_T is a previously established value). When K, T, and B are known, the remainder of the stresses in the roof truss may be obtained by drawing a stress diagram in the usual manner. The whole investigation is laborious and very often is omitted in practice. As previously stated, the sizes of the truss members, if conservatively proportioned, are seldom increased beyond those resulting from the usual truss design.

367. Typical Design Example.

In order to summarize the foregoing notes on the design of typical mill buildings, an illustrative problem will be worked out for the following data:

Length of building = 150′-0″ center to center of end frames = 10 bays at 15′-0″ each. Width of building = 60′-0″ center to center of outside columns. Height to eaves = 18′-0″. One-quarter pitch roof. Monitor 7′-6″ high, 15′-0″ wide, to be in inside 8 bays. Roof and sides covered with

* The girts, in resisting wind pressure, also induce bending in the columns. Those located below the knee brace connection are effective. For buildings of ordinary heights, this may be neglected. For heights exceeding 30′-0″ to the eave, the amount of bending may be approximated by considering the column as a fixed-end beam with a series of concentrated loads.

corrugated sheeting. Windows in each side, 7′-3″ × 10′-6″. Windows in ends, 6′-3″ × 10′-6″. One door in each end, 3′-6″ × 7′-0″. Two doors in each side, 7′-6″ × 10′-0″ (located in third bay from each end). Columns to be supported by reinforced concrete footings. Knee braces connected to columns 3′-9″ below trusses. Total combined load on roof =

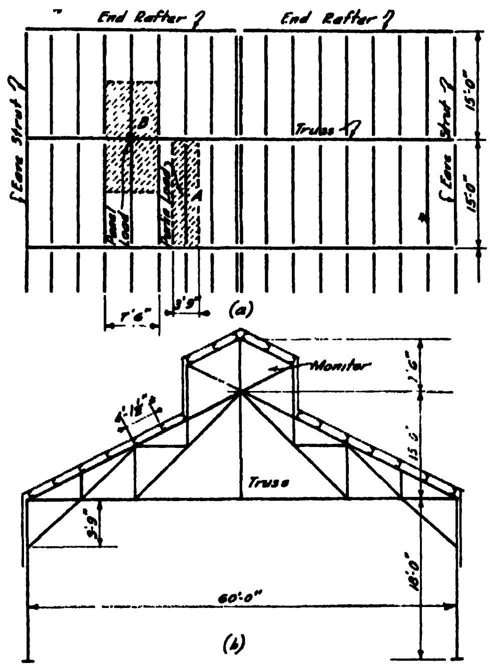


FIG. 578

40#/□′ (dead + live), all considered to act vertically (Art. 161), — this includes local action of wind on roof. Wind pressure (for design of bracing, etc.) = 25#/□′. Use type of truss shown in Fig. 578 (b). Draw typical line diagrams of transverse bent, side framing, end frame, top chord bracing, and bottom chord bracing, showing all the members and their sizes.

Distance between panel points of truss

$= \sqrt{(7.5)^2 + (3.75)^2} = 8.21'$ in plane of top chord.

This is too great a distance for corrugated sheeting to span. Use intermediate purlins as shown in Fig. 578 (b).

From Art. 164, use #18 gauge corrugated sheeting.

From Fig. 578 (a), load on purlin = 3.75 × 40 = 150#/ft.

$$\frac{I}{c} = \frac{1.5 \times 150 \times (15)^2}{16{,}000} = 3.17''^3$$

$$\text{Try } 6 \sqsubset 8.2 \quad \frac{I}{c} = 4.3''^3$$

$$I = 13.0''^4$$

$$D = \frac{5\,W \cdot l^3}{384\,E \cdot I} = \frac{5(150 \times 15) \times 15 \times 15 \times 15 \times 12 \times 12 \times 12}{384 \times 29{,}000{,}000 \times 13.0} = 0.45''$$

$$D \text{ (allowable)} = \frac{l}{360} = \frac{15 \times 12}{360} = 0.5'' \quad \text{O.K.}$$

Use 6 ⊏ 8.2 purlins.

In Fig. 578 (b), spacing of truss panels = 7′-6″ horizontally. Panel load = 7.5 × 15.0 × 40 = 4500#.

Figure 579 (a) shows the truss diagram and (b) shows a graphical solution for the stresses. Figure 579 (c) and (d) shows the resulting stresses in the monitor frame. Figure 580 gives the kinds and the values of the stresses in a transverse

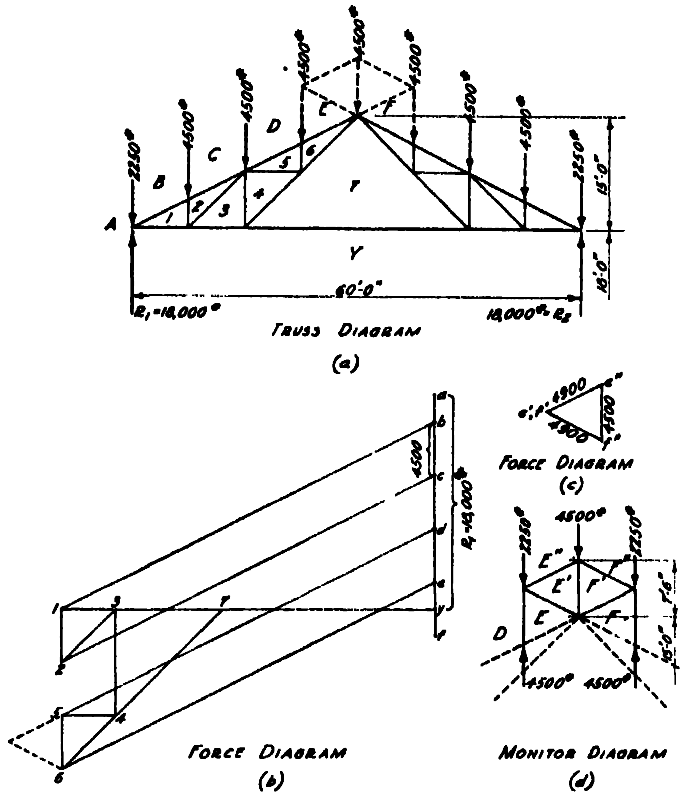


FIG. 579

bent. The design of the truss members will be considered after other factors are established.

Although the sizes of the bracing members are often established arbitrarily, the theoretical values will be developed.

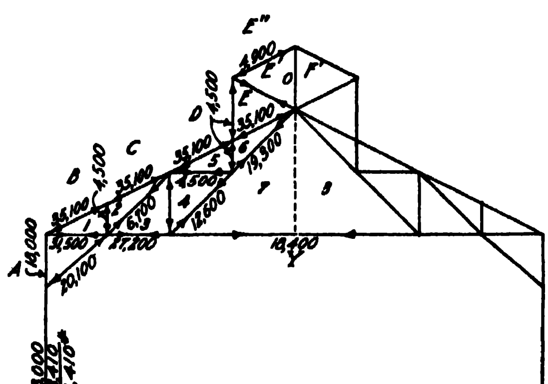


FIG. 580

Figure 581 (a) shows a partial elevation of the end frame, and the assumed areas tributary to the "panel points" of the top and bottom chord bracing, and at the foot of each column.

$$\text{Force at } a = \frac{9.0 + 10.92}{2} \times 7.5 \times 25\#/\square' = 1870\#$$

$$b = \frac{10.92 + 14.62}{2} \times 15.0 \times 25 = 4780$$

$$c = 15.58 \text{ (ave.)} \times 15.0 \times 25 = 5840$$

$$d = 7.5 \times 9.0 \times 25 = 1690$$

$$e = 15.0 \times 9.0 \times 25 = 3380$$

$$f = 15.0 \times 9.0 \times 25 = 3380$$

$$g = \frac{7.5 \times 1.92}{2} \times 25 = 180$$

$$h = \frac{1.92 + 5.62}{2} \times 15.0 \times 25 = 1420$$

$$i = 6.58 \text{ (ave.)} \times 15.0 \times 25 = 2470$$

Figure 581 (*b*), (*c*) and (*d*) shows these loads.

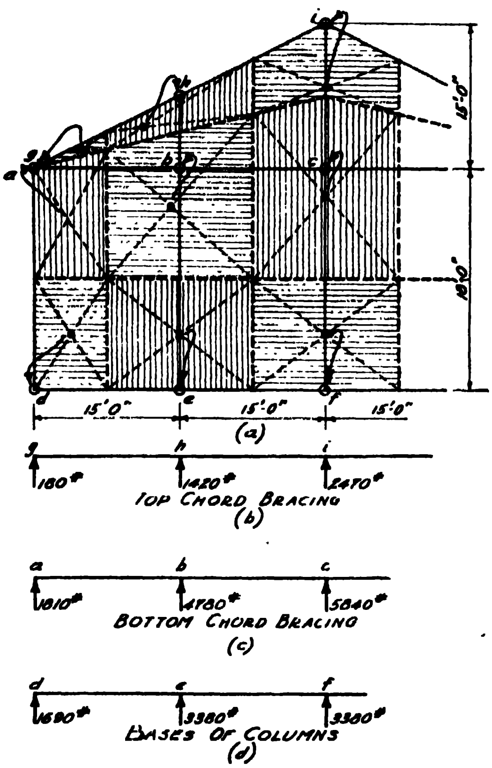


FIG. 581

Figure 582 illustrates the stress solution for the bottom chord bracing "truss." That in (*a*) is for the wind on the outside of the building at one end, and (*b*), for the loads brought to the same "truss" through the frame, when the wind is blowing in the opposite direction. The stress diagram for the latter would be similar to that shown in (*a*). The diagonals are assumed to carry tension only.

Maximum tension (in member 1–2) = 10,900#

$$\text{Net area required} = \frac{10{,}900}{16{,}000} = 0.68\square''$$

Gross area of 1 ∟ $2\frac{1}{2} \times 2 \times \frac{1}{4}$ = 1.06□″
1 hole out = $\frac{7}{8} \times \frac{1}{4}$ = 0.22

Net area = 0.84□″ O.K.

Use $2\frac{1}{2} \times 2 \times \frac{1}{4}$ ∟ diagonals for bottom chord bracing.

Maximum compression (in member 4–5) = 5840#

$L = 15'\text{-}0''$. Max. $\frac{l}{r} = 200$. Min. $r = \frac{15 \times 12}{200} = 0.9''$

From Table 78, Try 2 ∟ $3 \times 2 \times \frac{1}{4}$ long legs vertical.

$r(\text{min.}) = 0.95''$ $p = 16{,}000 - \frac{70 \times 15 \times 12}{0.95} = 2700\#/\square''$

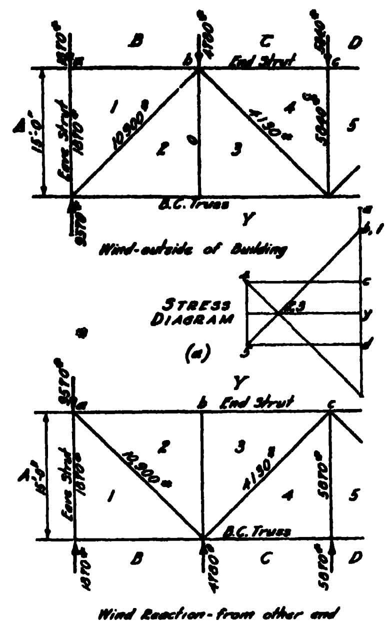


FIG. 582

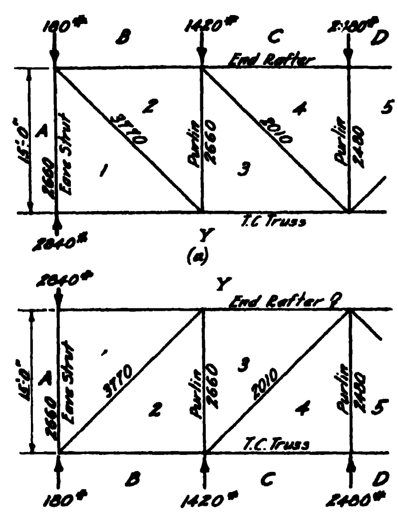


FIG. 583

$$\text{Area required} = \frac{5840}{2700} = 2.16\square''$$

Area of 2 ∟ $3 \times 2 \times \frac{1}{4}$ = 2.38□″ O.K.

Use 2 ∟ $3 \times 2 \times \frac{1}{4}$ for struts in bottom chord bracing.

Figure 583 shows the resulting stresses in the top chord bracing. This bracing is planned to be in 4 transverse panels

as shown, running between adjacent top chords. When the purlins are farther apart, the bracing is often framed between purlins instead. The stresses may be obtained by drawing a diagram similar to Fig. 582 (*a*), or by direct computations, as follows:

$$\text{Stress 1–2} = (2840 - 180) \times \frac{21.25}{15.0} = 3770\#$$

$$\text{Stress 3–4} = 1420 \times \frac{21.25}{15.0} = 2010\#$$

Maximum tension (in member 1–2) = 3770#

$$\text{Net area required} = \frac{3770}{16{,}000} = 0.24\square''$$

Net area of ¾″ φ rod = 0.302□″ O.K.

Use ¾″ φ rods for diagonals in top chord bracing.

The next step is to investigate the bracing in the side and end frames. Figure 584 illustrates the tributary areas in these cases.

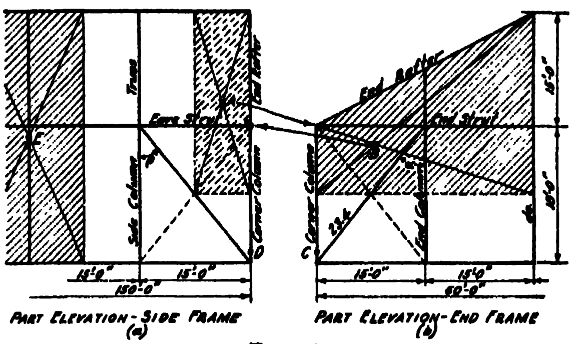


FIG. 584

Pressure on End Strut = wind load on area "*A*" in Fig. 584 (*a*) $= \frac{15.0}{2} \times \left(15.0 + \frac{18.0}{2}\right) \times 25\#/\square' = 4500\#$

$L = 15'\text{-}0''$. Max. $\frac{l}{r} = 200$. Min. $r = \frac{15 \times 12}{200} = 0.9''$

Try 2 ∟ 2½ × 2 × ¼ starred (Art. 271) $r = 2 \times 0.59 = 1.18''$

$$p = 16{,}000 - \frac{70 \times 15 \times 12}{1.18} = 5400\#/\square''$$

$$\text{Area required} = \frac{4500}{5400} = 0.83\square''$$

Area available = 2.12□″ O.K.

Use 2 ∟ 2½ × 2 × ¼ starred for end struts.

Diagonal bracing in end frame (2 panels).

$$\tfrac{1}{2} \text{ of force in end strut} = \frac{4500}{2} = 2250\#$$

$$\sec. \angle\, \alpha \text{ (Fig. 584 (b))} = \frac{23.4}{15.0} = 1.56$$

Stress in 1 diagonal, assuming any diagonal takes tension only, = 2250 × 1.56 = 3520#.

Use ¾″ φ rods for diagonals in end frame.

Pressure on Eave Strut = wind load on area "*B*" in Fig. 584 (*b*) $\left(\frac{15.0 \times 30.0}{2} + 30.0 \times \frac{18.0}{2}\right) \times 25\#/\square' = 12{,}380\#$

Similar to end strut. Area required $= \frac{12{,}380}{5400} = 2.29\square''$

Use 2 ∟ 2½ × 2 × 5/16 starred for eave struts.

Diagonal bracing in side frame. Assume wind force resisted by only 2 bays of diagonals, on account of possible expansion joints.

$$\tfrac{1}{2} \text{ of force in eave strut} = \frac{12{,}380}{2} = 6190\#$$

$$\sec \angle\, \beta \text{ (Fig. 584 (a))} = \frac{23.4}{15.0} = 1.56$$

Stress in 1 diagonal = 6190 × 1.56 = 9660#

$$\text{Net area required} = \frac{9660}{16{,}000} = 0.59\square''$$

Use 1″ φ rods for diagonals in side frame.

The horizontal components of the stresses in the diagonal bracing in the side and end frames are provided for by the shearing resistance of the anchor bolts of the column base.

Horizontal component of 9660# (max. diagonal stress)

$$= 9660 \times \frac{15.0}{23.4} = 6200\#$$

$$\frac{6200}{10{,}000} = 0.62\square'' \text{ area required.}$$

2-¾″ φ anchor bolts are ample for shear resistance (see later computations).

The anchor bolts provide shear resistance, but local bending is induced in the columns. Theoretically, a bottom strut is required to offset this, located at *C* and *D* in Fig. 584, but in practice a girt placed near these points will be sufficient for all ordinary cases.

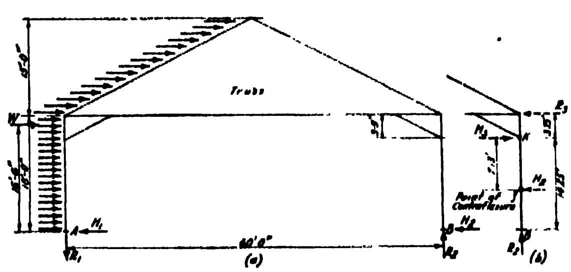


FIG. 585

The next step is to study the effect of the wind on a transverse bent. Figure 585 shows the general action. The total height = 18′-0″ + 15′-0″ = 33′-0″. Let W = the total wind force on one bent. This is the area "*E*" in Fig. 584 (*a*) times the unit wind pressure, or $W = 33.0 \times 15.0 \times 25\#/\square' = 12{,}400\#$. The direct stress in the side columns due to the overturning action may now be computed. Taking moments about *B* in Fig. 585,

$$60\, R_1 = 12{,}400 \times \frac{33.0}{2}, \text{ or } R_1 = 3410\#$$

$$\text{By } \Sigma H = 0, \quad R_1 = R_2 = 3410\#$$

The leeward column is the critical one (R_2), since extra compression is added to that caused by the superimposed load. R_1 induces tension in the windward column, which relieves the compression otherwise induced. The maximum uplift on a column is thus 3410# (neglecting the dead load on the column).

$$\text{Net area required for anchor bolts} = \frac{3410}{16{,}000} = 0.21\square''$$

Use 2-1″ φ anchor bolts for each column (see previous calculations).

The minimum number of anchor bolts should be two, and the minimum diameter 1″. They should be placed as far

apart as possible in the plane of the wind. Some engineers provide a size which will safely resist one and one-half times the bending moment at the base of the columns.*

Direct bending is caused in the windward column by the wind load. The load per ft. of height = 15 × 25 = 375#. Consider the portion of the column below the knee brace, or $L = 18.0 - 3.75 = 14.25$ ft. The wind load will be assumed as uniformly distributed.* The ends of this vertical "beam" will be assumed as fixed.

$$M = \frac{w \cdot L^2}{12} = \frac{375 \times (14.25)^2}{12} = 6350'\#.$$

This value does not control, however, and the bending moment induced in the column by the action of the knee brace is larger. This occurs at the foot of the knee brace at the leeward column.

If W (as calculated above) = 12,400#, then by $\Sigma H = 0$, $H_1 = H_2 = 6200\#$, in Fig. 585.

The point of contraflexure will be assumed as half-way between the foot of the knee brace and the base of the column (fixed end column). For purposes of computing the bending moment, H_2 may be assumed as acting at the point of contraflexure. (Fig. 585 (b)). Its arm with respect to the foot of the knee brace is then $\frac{1}{2}(18.0 - 3.75) = 7.13'$. On this basis,

$$M = H_2 \times \text{arm} = 6200 \times 7.13 = 44{,}200'\#$$

The side columns must be proportioned for this bending, in addition to the direct compression induced by the superimposed loads (see later computations).

The next step is to determine the stress in the knee brace, induced by the wind. Considering the leeward column as a free body, as in Fig. 585 (b), assuming H_2 to be at the point of contraflexure, and taking moments about K,

$$\begin{aligned} H_2 \times 7.13 &= R_3 \times 3.75 \\ 6200 \times 7.13 &= R_3 \times 3.75, \quad \text{or} \quad R_3 = 11{,}750\# \\ \text{By } \Sigma H = 0, \quad H_3 &= R_3 + H_2 \\ &= 11{,}750 + 6200 = 17{,}950\# \end{aligned}$$

H_3 is the horizontal component of the stress in the knee brace.

$$\sin \angle \text{ at } K = \frac{H_3}{\text{knee brace}}, \text{ or stress in knee brace} = \frac{H_3}{\sin \angle}$$

$$\text{stress} = 17{,}950 \div \frac{7.5}{8.4} = 20{,}100\#$$

A theoretical investigation may be made to determine the stresses in the truss induced by the stress in the knee brace. The first step is to find the stresses in the members Y–1 and B–1 (Fig. 580) due to such action (independent of the regular roof load).

By $\Sigma V = 0$, the vertical component of the stress in B–1 $= R_2 -$ the vertical component of the stress in the knee brace.

$$\angle \text{ at } K = \angle \tan^{-1} = \frac{7.5}{3.75} = 2.0, \quad \text{or} \quad \angle \text{ at } K = 63° \ 30'$$

$\cos 63°\ 30' = 0.446.$

Vertical component of B–1 = 3410 − 20,100 × 0.446 = 5570#.

Stress in B–1 = vertical component ÷ sine of the angle of inclination of the top chord, or = vertical component ÷ sine 26° 34′ ($\frac{1}{4}$ pitch roof) = 5570 ÷ 0.447 = 12,500#.

The stress in Y–1 = R_3 − the horizontal component of the stress in B–1 = R_3 − stress in B–1 × cos 26° 34′ = 11,750 − 12,500 × 0.894 = 580#.

With these two values established, and the stress in the knee brace as a force, the stresses in the remaining members may be found, usually by a graphical solution.†

The above investigation is not usually made in practice, as it is laborious, and many of the sizes of the truss members would not change, particularly when practical, minimum members are considered. However, some engineers prefer to arbitrarily increase the stresses in the truss members, say 20%, to make an allowance for the action of the knee brace. This will be done here. The following stresses will then result (refer to Fig. 580):

B–1, C–2, D–5, E–6	= 35,100 × 1.2 =	42,200#
Y–1	= 31,500 × 1.2 =	37,800
Y–3	= 27,200 × 1.2 =	32,600
Y–7	= 18,400 × 1.2 =	22,100
1–2, 4–5, 5–6, D–E	= 4,500 × 1.2 =	5,400
2–3	= 6,700 × 1.2 =	8,050
3–4	= 9,000 × 1.2 =	10,800
E–E', E'–E''	= 4,900 × 1.2 =	5,880
4–7	= 12,600 × 1.2 =	15,100
6–7	= 19,300 × 1.2 =	23,200

Refer to Fig. 586.

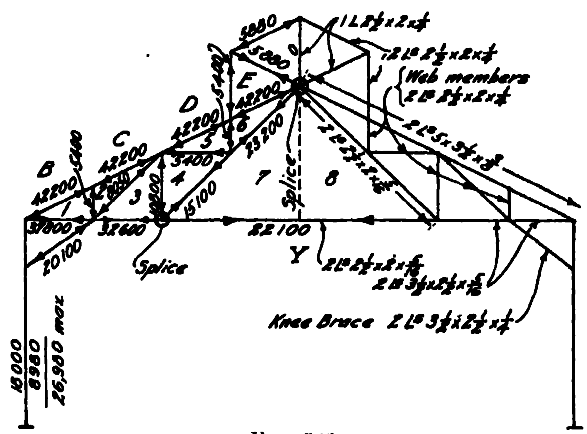


FIG. 586

Design of truss members. The top chord will be made in 1 length each side. Maximum stress = 42,200#. L = 8′-3″ ± between joints.

$$\text{Min. } r = \frac{8.25 \times 12}{120} = 0.825''. \quad \text{Assume gusset plates } \tfrac{3}{8}''$$

thick, average. Try 2 ∟ 4 × 3, min. r = 1.27″ (ave.)

$$p = 16{,}000 - \frac{70 \times 8.25 \times 12}{1.27} = 10{,}540\#/\square''$$

$$\text{Area required} = \frac{42{,}200}{10{,}540} = 4.00\square''$$

This requires 2 ∟ 4 × 3 × $\frac{5}{16}$

However, the member is subjected to indirect stress, caused by the bending due to an intermediate purlin load of 150 × 15 = 2250#.

$$\text{Assuming fixed ends, } M = \frac{P \cdot L}{8}$$

$$M = \frac{2250 \times 8.25}{8} \times 12 = 27{,}800''\#$$

$$\text{Try 2 ∟ } 4 \times 3 \times \tfrac{3}{8}, \ \frac{I}{c} = 2 \times 1.5 = 3.0''^3, \ A = 4.96\square''$$

* In reality, the wind load is a series of concentrations brought in by the girts. However, uniform load distribution is sufficiently accurate for this investigation and, in some cases, gives a larger bending moment which would be on the safe side.

† Such a type of solution is discussed in M. S. Ketchum's "Mill Buildings," — McGraw-Hill Book Co.

$$\text{Indirect stress} = \frac{27{,}800}{3.0} = 9{,}270$$

$$\text{Direct stress} = \frac{42{,}200}{4.96} = 8{,}500$$

$$\text{Total} = 17{,}770\#/\square'' \quad \text{excessive}$$

Try 2 ∟ 5 × 3½ × ⅜, $\frac{I}{c} = 2 \times 2.3 = 4.6''^3$, $A = 6.10\square''$

$$\text{Indirect stress} = \frac{27{,}800}{4.6} = 6{,}040$$

$$\text{Direct stress} = \frac{42{,}200}{6.10} = 6{,}920$$

$$\text{Total} = 12{,}960\#/\square''$$

$$p = 16{,}000 - \frac{70 \times 8.25 \times 12}{1.46} = 11{,}240\#/\square''$$

Maximum allowable $= 1.25 \times 11{,}240 = 14{,}040\#/\square''$ O.K.

Use 2 ∟ 5 × 3½ × ⅜ for top chord.

Bottom chord. The member Y–7 will be shipped loose, with the left and right hand remainders as shipping pieces. Maximum tension = 37,800#.

$$\text{Net area required} = \frac{37{,}800}{16{,}000} = 2.36\square''$$

Try 2 ∟ 3½ × 2½ × 5/16, gross area = 3.56

2 holes out, each angle,

$$= 2 \times 2 \times \tfrac{7}{8} \times \tfrac{5}{16} = 1.09$$

2.47□″ net O.K.

Use 2 ∟ 3½ × 2½ × 5/16 for Y–1 and Y–3.

Member Y–7. Maximum tension = 22,100#

$$\text{Net area required} = \frac{22{,}100}{16{,}000} = 1.38\square''$$

Try 2 ∟ 2½ × 2 × 5/16, gross area = 2.62

2 holes out, each angle,

$$= 2 \times 2 \times \tfrac{7}{8} \times \tfrac{5}{16} = 1.09$$

1.53□″ net O.K.

Use 2 ∟ 2½ × 2 × 5/16 for Y–7.

Members 6–7, and 4–7, make in 1 length
Max. stress = 23,200#

Use 2 ∟ 2½ × 2 × 5/16 for X–7.

Web members. Maximum compression = 10,800# (= 3–4).

$$L = 7'\text{-}6''. \quad \text{Min. } r = \frac{7.5 \times 12}{120} = 0.75''$$

Try 2 ∟ 2½ × 2 × ¼, min. r = 0.78

$$p = 16{,}000 - \frac{70 \times 7.5 \times 12}{0.78} = 7940\#/\square''$$

$$\text{Area required} = \frac{10{,}800}{7940} = 1.36\square''$$

Area available = 2.12□″ O.K.

Use 2 ∟ 2½ × 2 × ¼ for 3–4.
Make 1–2 and 5–6 the same
(the minimum desirable section).

Members 2–3 and 4–5. Maximum tension = 5,400#

Use 2 ∟ 2½ × 2 × ¼

For members D–E and E'–E'', use 2 ∟ 2½ × 2 × ¼.
For members E–E', E'–F', and tie 7–8, use 1 ∟ 2½ × 2 × ¼.
Knee brace. Maximum stress = 20,100#

$$L = 8'\text{-}3''\pm. \quad \text{Min. } r = \frac{8.25 \times 12}{120} = 0.825''$$

Try 2 ∟ 3½ × 2½, min. r (ave.) = 1.11″

$$p = 16{,}000 - \frac{70 \times 8.25 \times 12}{1.11} = 9770\#/\square''$$

$$\text{Area required} = \frac{20{,}100}{9770} = 2.06\square''$$

Area 2 ∟ 3½ × 2½ × ¼ = 2.88□″ O.K.

Use 2 ∟ 3½ × 2½ × ¼ for knee braces.

Side columns. From roof = 18,000
Vertical component of stress in knee brace = 8,980

Maximum load = 26,980#

Maximum bending = 44,200′# (see previous calculations).
Unbraced length = 14′-3″ = 14.25′.

$$\text{Minimum radius of gyration} = \frac{14.25 \times 12}{120} = 1.43''$$

$$\text{Trial area} = \frac{26{,}980}{10{,}000} = 2.70\square''$$

Try 4 ∟ 3½ × 2½ × ¼ laced, 12″ back to back.

From properties of sections, $A = 5.76\square''$

$$\frac{I}{c} = \frac{170}{6} = 28.3''^3 \qquad I_{1-1} = 170''^4$$

$$I_{2-2} = 12.7''^4$$

$$r_{2-2}\ (\text{min.}) = 1.76''$$

$$p = 16{,}000 - \frac{70 \times 14.25 \times 12}{1.76} = 9240\#/\square''$$

$$\text{direct stress} = \frac{26{,}980}{5.76} = 4780$$

$$\text{indirect stress} = \frac{44{,}200}{28.3} = 1560$$

6340#/□″ O.K.

Side columns
Use 4 ∟ 3½ × 2½ × ¼ Laced
2¼ × ¼ Lacing.

Load on end columns = 15.0 × 7.5 × 40 = 4500# from roof
From end frame = 15.0 × 21.25 × 15 = 4780

9280#

$$\text{Min. } r = \frac{18.0 \times 12}{120} = 1.80''$$

Column braced in sidewise direction by girts
$r_{1-1} = 2.86''$ for 7 I 15.3

$$= 16{,}000 - \frac{70 \times 18 \times 12}{2.86} = 10{,}710\#/\square''$$

End columns
Use 7 I 15.3

$$\text{Load on corner columns} = \frac{9280}{2} = 4640\#$$

Braced both ways by girts.

Corner columns
Use ∟ 6 × 6 × ½.

Girts — maximum spacing = 3′-8″. L = 15′-0″
Wind pressure = 25#/□′ Ld./Ft. = 25 × 3.67 = 92#
$M = 1.5 \times 92 \times (15)^2 = 31{,}000''\#$

$$\frac{I}{c} = \frac{31{,}000}{16{,}000} = 1.94''^3$$

Use 4 ⊏ 5.4 girts
½″ φ sag rods at mid-span
Use #22 gauge corrugated siding.

Figures 587, 588, and 589 show typical line diagrams of the elevation of the end frame, partial elevation of the side framing, and part plans of the bottom chord and top chord bracing, respectively, with the relative locations and sizes of the members. The bracing in the top and bottom chord planes and in the side framing occurs in the 1st, 4th, 7th, and 10th bays.

Prob. 367b. Make a complete design for a mill building of the type of transverse section shown in Fig. 590. Length of building = 18 bays at 15'-0" each. Use corrugated sheets for ends and sides and for roof except at sawteeth. Total load = 40#/□' of roof acting vertically. Wind load (for bracing, etc.) = 20#/□'. No knee braces required and no bottom chord bracing is necessary. Calculate the bending in the outside columns due to the wind action. Draw line diagrams showing the locations and sizes of all members.

Prob. 367c. Make a complete design for a mill building of the type discussed in the above illustrative example with the following variations:

Length of building = 192'-0" center to center of end frames = 12 bays at 16'-0" each. Width of building = 56'-0" center to center of outside columns. Height to eaves = 16'-6". Pitch of roof 30°. Monitor 7'-0" high, 14'-0" wide, to be in inside 8 bays. Roof and sides covered with corrugated sheeting. Knee braces connected to columns 4'-0" below trusses. Allow for snow load of 20#/□' of roof surface, acting vertically. Allow 5#/□' for weight of steelwork and 2.0#/□' for weight of corrugated sheeting for roof. Assume wind load acts normal to roof, based upon a 30#/□' pressure on a vertical plane (see Art. 160). Draw typical line diagrams showing the locations and sizes of all members.

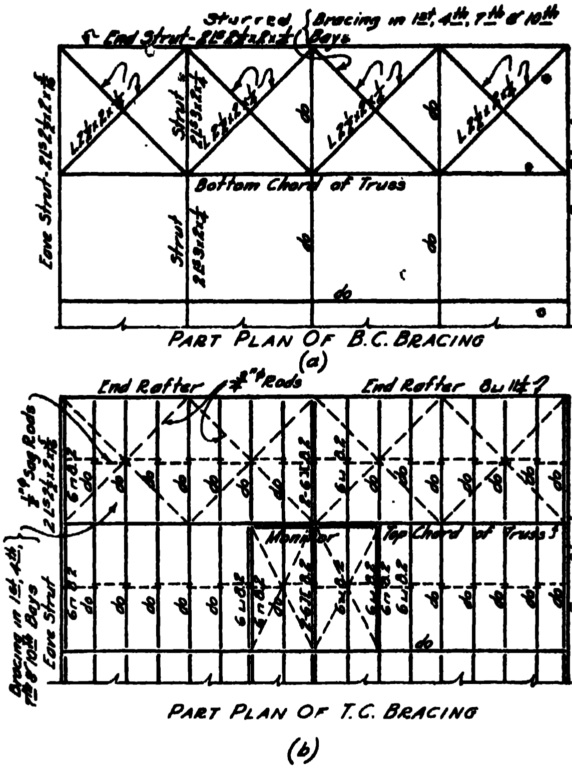


FIG. 589

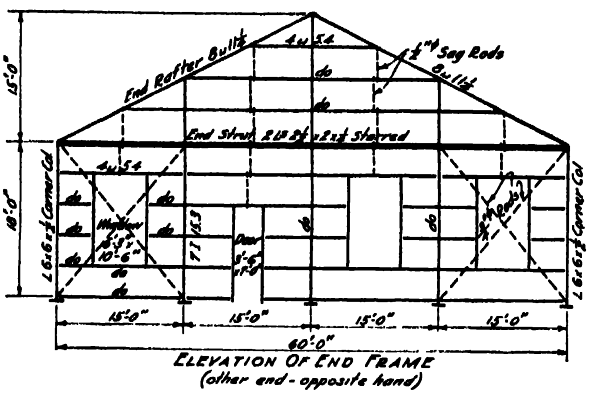


FIG. 587

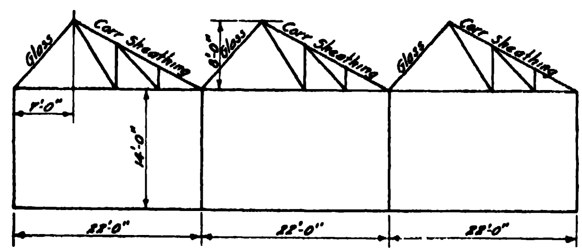


FIG. 590

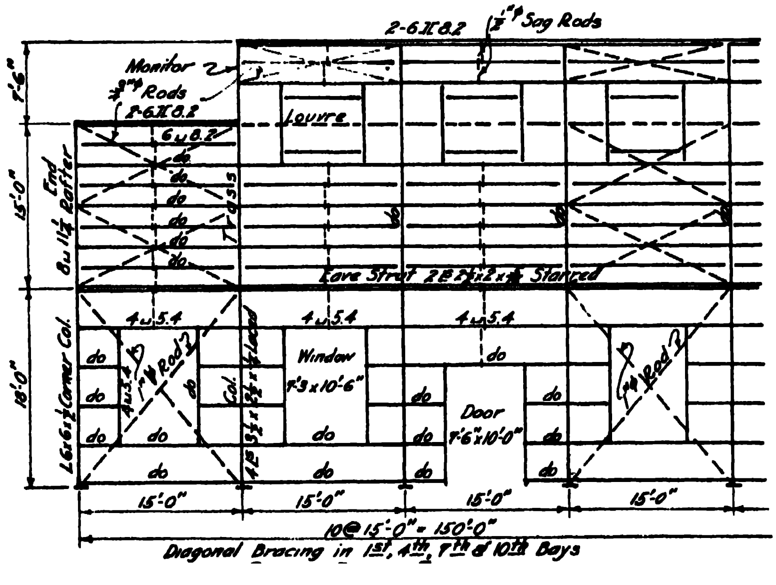


FIG. 588

368. "Standard" Buildings.

In recent years particularly, there has been a tendency to standardize certain types of buildings, especially mill structures, when the conditions surrounding the project do not impose

special requirements of size and irregular framing, and when standard equipment is to be installed for a definite kind of manufacture. There are a few companies which specialize in this kind of work.*

Advantages which are claimed are speed of erection, lower costs, and possibility of lateral extension by combinations of interchangeable units. This is made possible by a permanent stock of fabricated structural steel, roofing lumber, steel sash, and so on, held subject to prior sale and ready for immediate shipment. Standard designs, specifications, and plans, on a quantity production basis, are used in so far as they can generally be made to conform with the majority of building codes. Figure 591 illustrates typical structures of this kind. In these "Standard Factory Buildings," the walls are built of brick, clay tile, concrete block or concrete tile, which of course are more enduring than when a light covering is used.

Another type of standardized truss which has been recently introduced on the market is the Massillon arc-welded unit. Figure 592 shows the characteristic features of such members. The top chords are curved to an arc so that uniform compressive stresses result in them. The bottom chord is made level with the bearing points. All members are made of single angle sections, and the shop connections are electric arc-welded. Field connections are made with bolts. Standard lateral bracing is provided to connect to standard punchings in the trusses, and the top chords are punched to receive purlin connections or those for other secondary members, such as joists.

These trusses are made in two series of 5 trusses each, one designated as a "light load" series and the other as a "heavy load" group. These trusses are made to accommodate spans from 40'-0" to 60'-0"† by grouping a range of spans and loadings into usable limits of a given structural section. Small changes in span may be suited by extending the trusses

TABLE 93‡

MASSILLON STANDARD CURVED CHORD ROOF TRUSSES

Total Safe Loads in Lbs. per Sq. Ft.

Uniformly Distributed

Light Load Series

Truss Number	Clear Span	Bay width in feet (center to center of trusses)								
		12'	13'	14'	15'	16'	17'	18'	19'	20'
	40'-0"	74	68	63	58	55	52	49	47	44
44 CL.	41'-0"	72	66	62	56	54	51	48	46	43
	42'-0"	70	64	60	55	53	50	47	44	42
Weight	43'-0"	68	63	59	54	51	49	46	43	41
1350#	44'-0"	66	62	58	53	50	47	45	42	40
	44'-0"	72	67	62	58	55	51	48	47	44
48 CL.	45'-0"	71	66	61	57	54	50	47	45	43
	46'-0"	69	64	60	56	52	49	46	44	42
Weight	47'-0"	68	63	59	55	51	48	45	43	41
1600#	48'-0"	66	61	57	53	50	47	44	42	40
	48'-0"	72	66	62	58	54	51	48	46	43
52 CL.	49'-0"	70	65	60	56	53	50	47	45	42
	50'-0"	69	64	59	55	52	49	46	44	41
Weight	51'-0"	67	62	58	54	51	48	45	43	40
1900#	52'-0"	66	61	57	53	50	47	44	42	39
	52'-0"	71	66	61	57	54	50	48	45	43
56 CL.	53'-0"	70	65	60	56	53	49	47	44	42
	54'-0"	68	63	59	55	52	48	46	43	41
Weight	55'-0"	67	62	58	54	51	47	45	42	40
2150#	56'-0"	66	61	57	53	50	46	44	41	39
	56'-0"	70	65	60	57	53	50	48	45	43
60 CL.	57'-0"	69	64	59	56	52	49	47	44	42
	58'-0"	68	63	58	55	51	48	46	43	41
Weight	59'-0"	67	62	57	54	50	47	45	42	40
2350#	60'-0"	66	61	56	53	49	46	44	41	39

Heavy Load Series

Truss Number	Clear Span	Bay width in feet (center to center of trusses)								
		12'	13'	14'	15'	16'	17'	18'	19'	20'
	40'-0"	102	95	88	82	77	73	68	65	62
44 CH.	41'-0"	100	93	86	80	75	71	66	63	60
	42'-0"	98	90	84	78	73	69	65	62	59
Weight	43'-0"	95	88	82	76	71	67	63	60	58
1700#	44'-0"	93	86	80	74	69	66	62	59	56
	44'-0"	106	99	92	86	80	76	71	67	64
48 CH.	45'-0"	105	97	90	84	78	74	69	66	63
	46'-0"	102	94	88	82	76	72	68	65	61
Weight	47'-0"	100	92	86	80	74	70	66	63	60
2200#	48'-0"	98	90	83	78	73	69	65	62	59
	48'-0"	100	93	86	80	75	71	67	64	60
52 CH.	49'-0"	99	91	85	79	74	69	65	63	59
	50'-0"	97	89	83	78	73	68	64	61	58
Weight	51'-0"	95	87	81	76	71	67	63	60	57
2500#	52'-0"	93	85	79	75	70	66	62	59	56
	52'-0"	100	92	85	79	75	70	66	63	60
56 CH.	53'-0"	98	90	83	78	73	69	65	62	59
	54'-0"	96	88	82	77	72	68	64	61	58
Weight	55'-0"	94	87	80	75	71	67	63	60	57
2800#	56'-0"	92	85	79	74	69	65	62	59	56
	56'-0"	97	89	83	78	73	69	65	62	58
60 CH.	57'-0"	95	87	82	77	72	67	63	60	57
	58'-0"	93	86	80	75	70	66	62	59	56
Weight	59'-0"	91	84	79	74	69	65	61	58	55
3100#	60'-0"	90	83	78	72	68	64	60	57	54

Designed in accordance with American Institute of Steel Construction's 1923 Specifications.
Decrease loads 10% if American Bridge Company's Specifications are used.
For special loadings due to shaft hangers, heating equipment, monorail, or other conditions — special designs are necessary.

* One of the most prominent in this type of construction is the Austin Company, Engineers and Builders, Cleveland, Ohio.

† Trusses for 40'-0" spans are shipped in one piece, and trusses for 60'-0" spans are shipped in two lengths, suitable splices being provided for the erection of the latter.

‡ Massillon Steel Joist Co., Canton, Ohio.

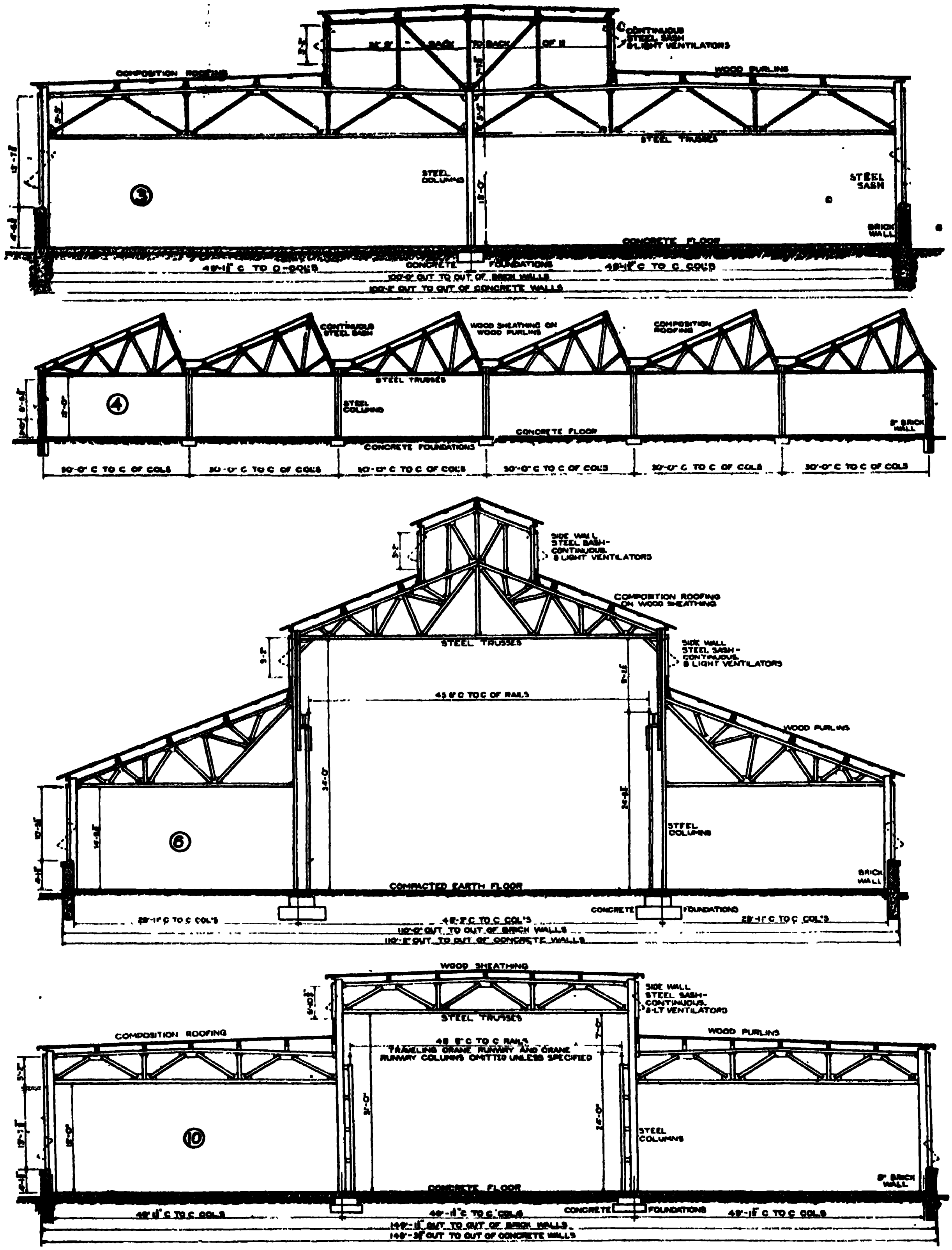


FIG. 591*

* Courtesy of the Austin Company.

beyond the walls to build into the eave construction, or by cutting small portions of the ends off, as the case may be. This is possible because the end gussets and bearing plates are especially designed for this purpose. Table 93 gives total safe loads, uniformly distributed, for this type of truss. By varying the spacing of trusses, any given uniform roof load capacity may be provided. Usually a L.L. of 25#/□′ of horizontal projection is sufficient for a curved roof of this kind, but the local building ordinance should be consulted. Concentrated loading conditions require special designs, as noted at the foot of the table. A suspended ceiling of metal lath and cement plaster may be used for appearance or to reduce the fire hazard.

ported by the building columns, as shown in Fig. 595. **Cranes are designated** by their capacity to lift loads **as light**, when 25 tons or less, **and as heavy**, when of larger capacity.

The design of crane girders becomes a special problem, because lateral forces are exerted upon them by the cross-travel of the load, impact, and in attempts to lift loads obliquely. Cranes are also subject to misuse. Some operators will attempt to drag loads out of the side wings of a building until they are under the crane and then lift them. This

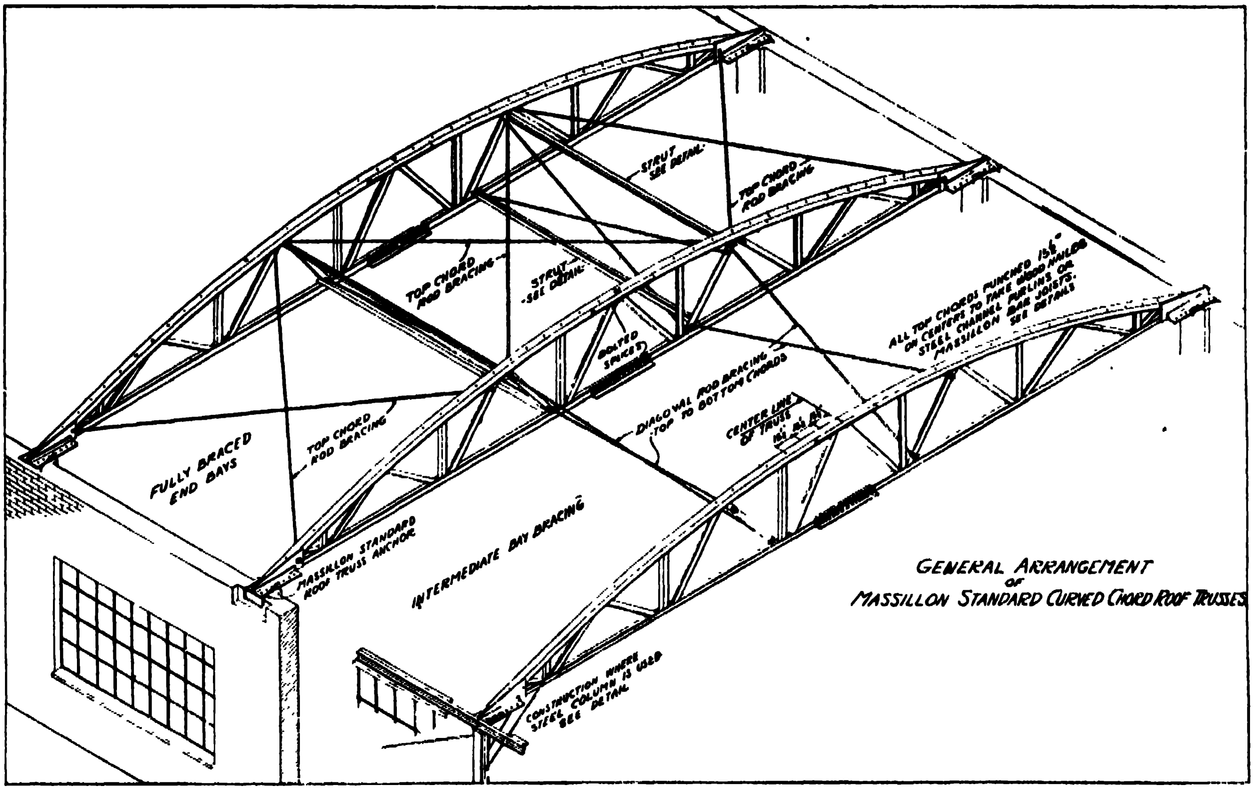


FIG. 592*

369. Crane Girders.

In the majority of mill buildings, travelling cranes are provided as a part of the equipment. Figure 593 illustrates a typical example. The design of the crane itself is made by the manufacturers, but it is necessary for the structural engineer to provide sufficient supports upon which the crane may function. The crane consists of a girder itself,† usually of varying depth, carrying a mechanism for lifting loads when the power is applied. The crane is supported at each end by end carriages which have two or four wheels each. These wheels run on rails provided on tops of the supporting girders (Fig. 594). The latter run lengthwise of the building and are sup-

* Massillon Steel Joist Co., Canton, Ohio.

† Care should be used in discriminating between the girders of the crane and the girders supporting the crane.

abuse results in the horizontal component of the diagonal lifting force tending to act as a lateral force upon a crane girder. Such a force also tends to distort the bent of framing where it occurs, so that mill buildings in which cranes are used must be designed conservatively. The crane girders themselves must also be designed for combined vertical and lateral bending.

The loads from the crane occur as wheel loads. Hence the criterions for the maximum bending moment and the maximum shear must be determined for moving loads. Figure 596 shows dimension diagrams for four and eight-wheel cranes and Table 94 gives the corresponding dimensions,‡ clearances,

‡ Values of P may be approximated when data are lacking by using the capacity in tons divided by 10, plus 7′-0″.

Fig. 593

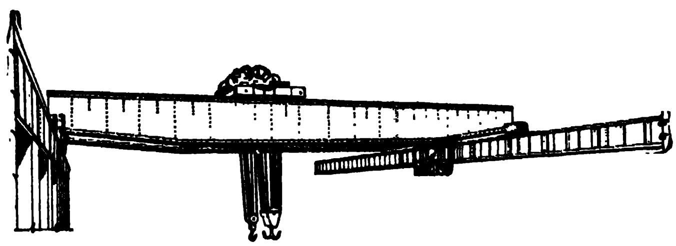
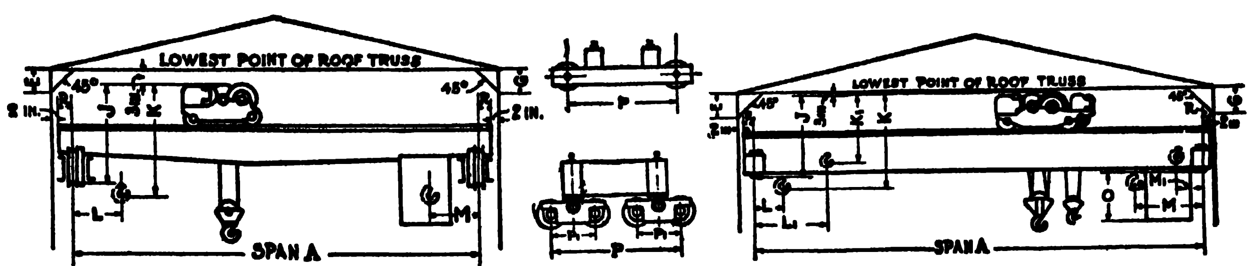


Fig. 594

Fig. 595



Dimension Diagram of Standard 4-wheel Shaw Cranes

Dimension Diagram of Standard 8-wheel Shaw Cranes

Fig. 596

TABLE 94*

DIMENSIONS OF STANDARD CRANES, 30-FT. HOIST (SEE FIG. [illegible])

A ft.	R in.	J ft. in.	K ft. in.	L ft. in.	M ft. in.	P ft. in.	Runway rail A.S.C.E., lbs. per yd.	Maximum load on each wheel, lbs.	A ft.	R in.	J ft. in.	K ft. in.	L ft. in.	M ft. in.	P ft. in.	Runway rail A.S.C.E., lbs. per yd.	Maximum load on each wheel, lbs.
				5-ton										15-ton			
30	8	4 8	4 6	3 3	2 2	9 0	40	11000	50	10	6 2	7 9	3 9	3 0	11 8	50	28000
40	8	5 2	4 6	3 3	2 6	10 0	40	13000	60	10	6 2	7 9	3 9	3 0	11 10	50	30000
50	8	5 2	4 6	3 3	2 6	10 0	40	14600	70	10	6 6	7 9	3 9	3 0	12 0	50	32000
60	8	5 6	4 6	3 3	2 6	10 10	40	16000	80	10	6 6	7 9	3 9	3 0	13 0	50	34000
80	8	5 6	4 6	3 3	2 6	13 0	40	20000	100	10	6 8	7 9	3 9	3 0	16 0	60	42000
				10-ton										20-ton			
40	8	5 6	5 0	3 7	2 7	11 0	40	20000	50	10	6 8	7 9	3 9	3 6	12 2	60	35400
50	8	5 8	5 0	3 7	2 7	11 2	40	21000	60	10	6 8	7 9	3 9	3 6	12 4	60	38000
60	8	5 8	5 0	3 7	2 7	11 4	40	22000	70	10	7 0	7 9	3 9	3 6	12 6	60	40800
80	10	6 3	5 0	3 7	2 7	13 0	50	27000	80	10	7 0	7 9	3 9	3 6	13 0	60	44000
100	10	6 3	5 0	3 7	2 7	16 0	50	34000	100	12	7 6	7 9	3 9	3 6	16 0	70	54000

A ft.	R in.	J ft. in.	K ft. in.	L ft. in.	K_1 ft. in.	L_1 ft. in.	M ft. in.	M_1 ft. in.	P ft. in.	P_1 ft.† in.	Runway rail A.S.C.E., lbs. per yd.	Maximum load on each wheel, lbs.
						30-ton, 5-ton Auxiliary						
50	12	7 6	9 0	6 6	4 0	7 2	6 0	2 10	13 0	.....	70	52000
60	12	7 8	9 0	6 6	4 0	7 2	6 0	2 10	13 2	.....	70	55000
70	12	7 8	9 0	6 6	4 0	7 2	6 0	2 10	13 4	.....	70	58400
80	12	8 0	9 0	6 6	4 0	7 2	6 0	2 10	13 8	.....	80	62600
100	12	8 0	9 0	6 6	4 0	7 2	6 0	2 10	16 0	.....	80	74000
						40-ton, 5-ton Auxiliary						
50	12	8 0	10 4	7 0	4 6	8 0	6 8	3 2	13 0	.....	80	65000
60	12	8 3	10 4	7 0	4 6	8 0	6 8	3 2	13 2	.....	80	60000
70	12	8 3	10 4	7 0	4 6	8 0	6 8	3 2	13 6	.....	90	74000
80	12	8 8	10 4	7 0	4 6	8 0	6 8	3 2	13 6	.....	90	80000
100	12	9 4	10 4	7 0	4 6	8 0	6 8	3 2	16 0	.....	90	92000
						50-ton, 10-ton Auxiliary						
50	12	8 9	11 6	7 9	4 8	8 6	7 2	3 3	13 6	.....	90	80000
60	12	9 0	11 6	7 9	4 8	8 6	7 2	3 3	13 6	.....	90	84000
70	12	9 0	11 6	7 9	4 8	8 6	7 2	3 3	14 9	.....	90	89000
80	18	10 4	11 6	7 9	4 8	8 6	7 2	3 3	13 6	4 6	70	50000
100	18	11 0	11 6	7 9	4 8	8 6	7 2	3 3	14 6	4 8	70	58000
						75-ton, 10-ton Auxiliary						
50	18	10 10	12 6	8 6	4 6	9 0	7 6	3 0	13 6	4 6	70	56000
60	18	11 2	12 6	8 6	4 6	9 0	7 6	3 0	13 6	4 6	80	60000
70	18	11 6	12 6	8 6	4 6	9 0	7 6	3 0	13 6	4 6	80	66000
80	18	12 0	12 6	8 6	4 6	9 0	7 6	3 0	13 6	4 6	90	74000
100	18	12 9	12 6	8 6	4 6	9 0	7 6	3 0	15 0	5 0	90	80000
						100-ton, 15-ton Auxiliary						
60	18	11 4	12 9	9 0	4 6	9 0	7 6	3 0	13 6	4 6	90	76000
80	18	12 6	12 9	9 0	4 6	9 0	7 6	3 0	13 0	4 6	90	86000
100	18	13 0	12 9	9 0	4 6	9 0	7 6	3 0	15 0	5 0	100	96000
						125-ton, 15-ton Auxiliary						
60	18	12 0	13 8	10 6	4 9	9 9	8 6	3 6	13 6	4 6	100	92000
80	18	13 0	13 8	10 6	4 9	9 9	8 6	3 6	13 6	4 6	100	100000
100	18	14 0	13 8	10 6	4 9	9 9	8 6	3 6	15 0	5 0	110	112000
						150-ton, 15-ton Auxiliary						
60	20	12 6	13 8	10 6	4 9	9 9	8 6	3 6	13 6	4 6	100	106000
80	20	13 6	13 8	10 6	4 9	9 9	8 6	3 6	14 0	5 0	150	118000
100	20	16 0	13 8	10 6	5 0	10 0	9 0	4 0	16 0	5 0	150	134000

* Shaw Cranes — Manning, Maxwell & Moore, Inc.

† These dimensions apply to 8-wheel cranes, and are the wheel-bases of the equalizing trucks.

and wheel loads. The action of cranes is accompanied by vibration and impact, and 25% (the usual specification) should be added to the weights of the moving parts and loads as tabulated to obtain the design loads.* The clearances given are usually of more interest to the mechanical engineer than the structural engineer, although some of them are important to the latter. The dimension J influences the elevation of the crane rails, and A, the distance between the centers of crane girders and consequently the final location of the supporting columns. Dimensions E and G affect limitations upon knee braces for the roof trusses. Wheel loads will vary slightly with different makes of cranes, but the above table gives average values. The height of hoist should be noted, which is usually a maximum of 30′-0″ or 32′-0″. Higher hoists may increase wheel bases, and consequently the loads, to some extent. Auxiliary cranes, as noted in the table, are usually employed with large cranes. The dimensions R and J may be reduced slightly if necessary. It should be noted that the clearances L and M are not equal, as the hook is not in the middle of the trolley.

The lateral loads are assumed to constitute 20% of the capacity of the crane, and to be distributed equally to each wheel. Thus, when there are four wheels, one-quarter is considered to act at each point. The same criterions for the positions of the loads apply as for the vertical wheel loads.† The top flange is the critical part of a crane girder, as additional compression is caused by the lateral loads. It is common practice to use additional metal in the top flange for this reason. Figure 597 shows some common types of sections used for crane girders. Bethlehem girder beams alone (Art. 3) may be

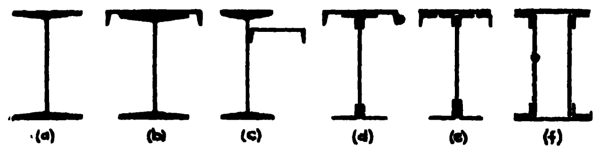


Fig. 597

used for light cranes. Their wide flanges offer considerable lateral resistance. Channels may be riveted to the top flanges to supply additional area and sidewise resistance if necessary, or the section in (c) may be used. These save considerable fabrication. For large cranes, built-up sections, such as shown in Fig. 597, are usually required. The breadth of flange should not be less than $\frac{1}{20}$ of the span. Table 95, which refers to Fig. 598, gives properties of I-beam crane girders which may save time in making computations. There is usually no opportunity to

TABLE 95

PROPERTIES OF I-BEAM CRANE GIRDERS WITH LATERAL STIFFENING CHANNELS‡

Axis 1-1				Axis 2-2			
y_1	y_2	y_3	I	x_1	x_2	x_3	I
4.46	3.54	1.54	61.21	2.965	5.17	1.035	32.96
4.547	3.453	1.453	62.35	3.595	6.54	0.405	71.20
5.04	3.96	1.96	92.38	3.03	5.28	1.30	35.26
5.22	3.78	1.78	95.29	3.94	7.37	0.39	104.47
5.16	3.84	1.84	94.25	3.60	6.71	0.73	75.06
5.60	4.40	2.40	133.84	3.085	5.40	1.575	37.80
5.76	4.24	2.24	136.87	3.635	6.85	1.025	78.90
5.89	4.11	2.11	139.84	4.265	8.22	0.395	147.70
6.91	5.09	3.09	245.81	3.615	7.06	1.385	84.70
7.09	4.91	2.91	251.96	4.175	8.50	0.825	156.75
7.31	4.69	2.69	259.35	4.945	9.73	0.055	276.50
8.54	6.46	4.46	507.02	3.645	7.31	1.855	93.65
8.775	6.225	4.225	521.65	4.125	8.83	1.375	170.50
9.07	5.93	3.93	539.30	4.775	10.18	0.725	299.10
9.58	5.42	3.42	572.10	6.135	11.82	0.635	654.00
10.115	7.885	5.885	911.00	3.73	7.50	2.27	103.08
10.39	7.61	5.61	938.40	4.14	9.09	1.86	183.30
10.73	7.27	5.27	972.90	4.70	10.53	1.30	319.20
11.38	6.62	4.62	1038.8	5.96	12.27	0.04	698.3
11.10	8.90	6.90	1327.6	3.765	7.61	2.485	111.72
11.395	8.605	6.605	1367.4	4.115	9.26	2.135	194.17
11.75	8.25	6.25	1417.5	4.625	10.75	1.625	334.90
12.46	7.54	5.54	1516.2	5.765	12.61	0.485	731.30
13.18	10.82	8.82	2348.0	4.035	7.715	2.965	128.12
13.50	10.50	8.50	2417.0	4.34	9.41	2.66	212.85
13.91	10.09	8.09	2506.3	4.78	10.97	2.22	358.05
14.75	9.25	7.25	2684.4	5.81	12.94	1.19	772.90

* An engineer should make sure that the tabulated wheel loads for a given type of crane do not include an allowance for impact. If they do, of course no 25% increase has to be made.

† The maximum lateral bending moment may be figured in proportion to the ratio of the two loads if desired.

‡ From article by J. H. Sawkins, Engineering News Record, Jan. 6, 1921.

stay the top flanges sidewise except at the columns, unless a stiffening girder (Art. 85) is used. If two girders, or a box girder as in Fig. 597 (*f*) is used, the two parts may be kept a fixed distance apart by means of steel diaphragms.

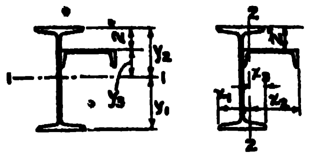


FIG. 598

The design of a crane girder must be made by "cut and try" methods. A preliminary design may be based upon the vertical wheel loads to obtain approximate sizes. When the bottom flange is established on this basis, 20% may be added to the gross area to determine a trial section. The fibre stresses in the two rectangular directions may then be calculated for the vertical and horizontal moments and the maximum combined stress found. This should not exceed the usual allowable by more than 25%, — this being a common specification.* When cover plates are used, they should extend the full length of the girder in order to provide a support at a given elevation for the rails. Countersunk rivets must be used at such points. The maximum lateral shear should be calculated and the end connections made strong enough to resist it. Substantial diaphragms should be used to connect the crane girders to the upper sections of columns, as shown in Fig. 599. This prevents the tendency toward overturning which is caused by the lateral forces. While the rail is not a part of the top flange material, it possesses a considerable amount of strength in bending and serves to distribute wheel loads. A common assumption when calculating the rivet spacing in the vertical legs of the flange angles is that each wheel load is distributed over a distance equal to the depth of the crane girder, with 30″ as a maximum. The usual rules for stiffeners for plate girders (Art. 57) apply here.

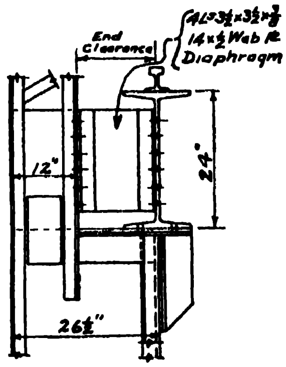


FIG. 599

When footwalks, stairways from the cage, transverse shafts, or bridge motor bases, are attached to the crane girder, the bending in a sidewise direction should be provided for, as some torsion is introduced (Fig. 600). Recommendations for the shears induced are discussed in Art. 85. The rivets through the top plate should be carefully investigated.

* Some designers prefer not to add any allowance for impact to the wheel loads (usually 25%, as previously discussed) and use the ordinary maximum allowable fibre stresses without increasing them. These practically offset each other, but not exactly in a theoretical sense, as the lateral forces are involved in the computations, which do not have any impact allowance added to them.

Illustrative Prob. 369a. Design a typical crane girder to support a 20 ton crane on a 60′-0″ span if the bents are 20′-0″ on centers. 30′-0″ hoist. Allow 25% for impact. Maximum tension 16,000#/□″ and maximum compression based on $16{,}000 - 150\frac{l}{b}$.

From Table 94, wheel base = 12′-4″
Wheel load = 38,000#
25% impact = 9,500

Maximum wheel load = 47,500#

Lateral Forces.
Crane capacity = 20 tons = 40,000#
20% of 40,000 = 8000# to 4 wheels.
8000 ÷ 4 = 2000# lateral force on each wheel.
From Table 94, side clearance = 10″ + 2″ = 12″
Top of rail to underside of trusses = 3″ + (6′-8″) = 6′-11″
60# A.S.C.E. rails to be used.

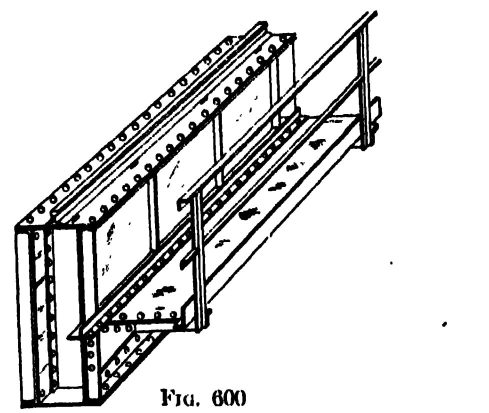


FIG. 600

The criterion for maximum bending moment for two moving loads is that the center of gravity of the loads should be as far to one side of the center-line of the span as the larger load is to the other. The center of gravity of two 47,500# loads is one-half way between them, so that for this case,

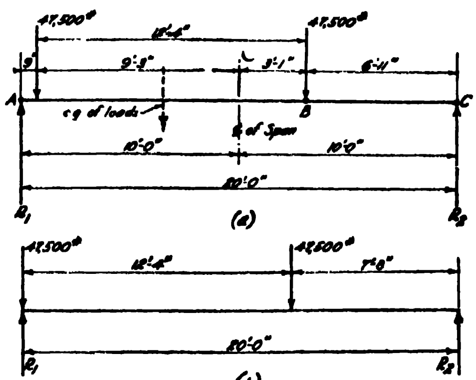


FIG. 601

one load is ¼ of 12′-4″ = 3′-1″ to the right of the center-line of span, as shown in Fig. 601 (*a*). Taking moments about *C*,

$$47{,}500 \times \frac{6.92}{20.0} = 16{,}440 \qquad 95{,}000$$

$$47{,}500 \times \frac{19.25}{20.0} = \underline{45{,}700} \qquad \underline{62{,}140}$$

$$62{,}140 = R_1 \qquad 32{,}860 = R_2$$

Point of 0 shear at *B*.

$M = 32{,}860 \times 6.92 = 228{,}000'\#$

When the wheel base is a large proportion of the span, one load at the center-line of span, with the other load off the span, may give a larger bending moment. For such a case,

$$M = \frac{P \cdot L}{4} = \frac{47{,}500 \times 20}{4} = 237{,}500'\# \text{ (controls)}$$

Assume dead load = 150#/ft.

$$M = \frac{w \cdot L^2}{8} = \frac{150 \times (20)^2}{8} = \underline{\quad 7{,}500}$$

Maximum vertical moment = 245,000′#

The bending moment due to lateral forces is obtained for the same position as for maximum vertical moment. Hence one wheel at mid-span is the governing condition.

$$M = \frac{P \cdot L}{4} = \frac{2000 \times 20}{4} = 10{,}000'\#$$

Usual maximum allowable compression =

$$16{,}000 - 150\frac{l}{b}. \quad \text{Assume } b = 12''.$$

$$16{,}000 - \frac{150 \times 20 \times 12}{12} = 13{,}000$$

Maximum allowable compression for combined stress

$= 1.25 \times 13{,}000 = 16{,}200\#/\square''$.

Try 24 BG 120. $\frac{I}{c}(1\text{-}1) = 300.6''^3$. $\frac{I}{c}(2\text{-}2) = 41.6''^3$

$$s_{(1-1)} = \frac{245{,}000 \times 12}{300.6} = 9{,}780$$

$$s_{(2-2)} = \frac{10{,}000 \times 12}{41.6} = \underline{2{,}890}$$

Combined stress = 12,670#/□″ O.K.

The 24 BG 120 is satisfactory, but possibly a smaller size may be used.

Try 20 BG 112. $\frac{I}{c}(1\text{-}1) = 234.2''^3$ $\frac{I}{c}(2\text{-}2) = 39.9''^3$

$$s_{(1-1)} = \frac{245{,}000 \times 12}{234.2} = 12{,}520$$

$$s_{(2-2)} = \frac{10{,}000 \times 12}{39.9} = \underline{3{,}000}$$

15,520#/□″ O.K.

Use 20 BG 112

Other sections might be used such as in Fig. 597 (*b*) or (*c*). Try section as in (*c*) with 24 I 80 and 10 ⊏ 15. Referring to Table 95 and Fig. 598

$I_{(1-1)} = 2417''^4$, $y^2 = 10.50''$ (compression fibres control),

$I_{(2-2)} = 212.85''^4$ and $x_2 = 9.41''$.

$$\frac{I_{(1-1)}}{c} = \frac{2417}{10.50} = 229''^3 \text{ and } \frac{I}{c}(2\text{-}2) = \frac{212.85}{9.41} = 22.6''^3$$

$$s_{(1-1)} = \frac{245{,}000 \times 12}{229} = 12{,}850$$

$$s_{(2-2)} = \frac{10{,}000 \times 12}{22.6} = \underline{5{,}310}$$

18,160# excessive

Try 24 I 80 and 12 ⊏ 20½

$$\frac{I_{(1-1)}}{c} = \frac{2506.3}{10.09} = 248.6''^3 \text{ and } \frac{I}{c}(2\text{-}2) = \frac{358.05}{10.97} = 32.6''^3$$

$$s_{(1-1)} = \frac{245{,}000 \times 12}{248.6} = 11{,}890$$

$$s_{(2-2)} = \frac{10{,}000 \times 12}{32.6} = \underline{3{,}680}$$

15,570#/□″ O.K.

Could use 24 I 80 and 12 ⊏ 20½ as in Fig. 597 (*c*).

Maximum vertical shear occurs when one load is practically at one end. Referring to Fig. 601 (*b*),

$$47{,}500 \times \frac{7.67}{20.0} = 18{,}200$$

$\underline{47{,}500}$

65,700#

L.L. = 65,700

D.L. of girder 10×112 = $\underline{1{,}120}$

Max. shear = 66,820#

Safe vertical shear on web of 20 BG 112 = 98,500#, or safe vertical shear on web of 24 I 80 (neglecting channel) = 120,000#.

By placing the 2000# lateral wheel forces in the same positions as in Fig. 601 (*b*),

$$2000 \times \frac{7.67}{20.0} = 770$$

$\underline{2000}$

2770# maximum horizontal shear.

The end connections must be designed to resist this force. For either larger spans or a heavier crane, crane girders made of rolled shapes may not be sufficient and a built-up section such as in Fig. 597 (*d*) may be necessary. When the maximum horizontal and vertical shears and moments are established, the design is similar to that of plate girders (Chap. 5).

Prob. 369b. Design a typical crane girder to support a 15 ton crane on a 50′-0″ span if the bents are 22′-0″ on centers. Follow the usual specifications. Select the most economical section (*a*) for a Bethlehem girder beam, (*b*) for a standard beam with a channel, as in Fig. 598.

Prob. 369c. Design a typical crane girder to support a 40 ton, 5 ton auxiliary crane on a 60′-0″ span if the bents are 20′-0″ on centers. Follow the usual specifications. Use section similar to Fig. 597 (*d*). Depth limited to 48″±. Refer to Chap. 5 for proportioning of parts.

370. Crane Girder Details.

There are a number of details which must be properly included with such work. The rails are usually made a part of the structural steel contract. Standard A.S.C.E. railroad sections are now quite universally used, and the size is made to vary with the crane, as shown in Table 94. The rails are fastened to the girder by means of steel **clips or hook bolts,** as shown in Fig. 602. The latter are used for narrow flanges, — chiefly for I beams — as the flanges are too narrow for clips. No punching is required in the flanges for these. For wide flanges, clips must be used. Both are spaced 2′-0″ o.c., and provide for adjusting small inaccuracies in the alignment of the rails. The rails are joined by standard flat bar or rolled **fish plates** which bear against the webs and bases, as shown in Fig. 602. Angle splices should not be used as they are apt to interfere with the double flanged crane wheels. Standard **crane stops** (bumpers) are provided at the ends of the last rails. These are usually a casting or a combination

of bent steel plates bolted in place, as shown in Fig. 602. All of the above fittings are standardized and vary with the size of rail, as indicated. Gear guards and track clearers are supplied by the crane company.

371. Crane Columns.

The design of the columns supporting the crane girders requires special considerations as far as the selection of a cross-section is concerned. The upper section (that above the crane girders) may be de-

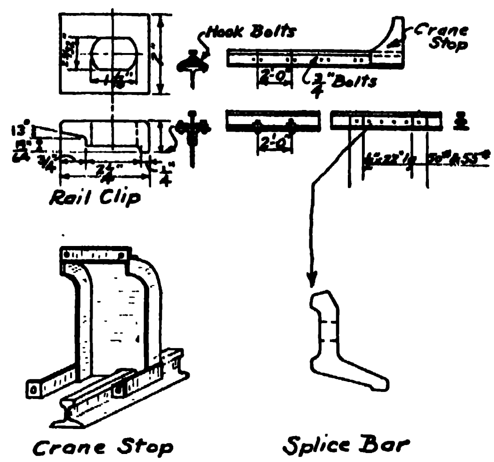


FIG. 602

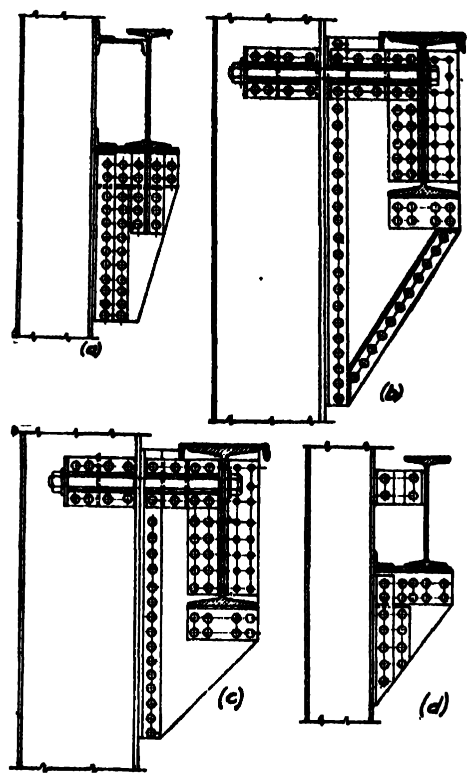


FIG. 603

signed in the usual manner (Arts. 242 and 248). When the width of the upper section is established, the width of the lower section must be made such that proper end clearance is provided for the crane girder. End clearances are given in Table 94 and

FIG. 604*

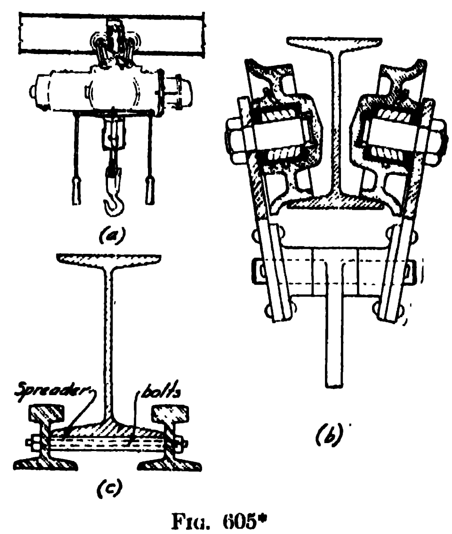


FIG. 605*

the method of allowing for them is illustrated in Fig. 599.

The crane girders are usually supported as shown in Fig. 599, by the use of seat angles and stiffeners combined with a partial column cap. The load from the crane is the maximum shear in the crane girder (similar to that computed in Illustrative Prob. 369*a*), making allowance for the dead weight of the crane girders (if only one crane is provided for). This load is eccentrically applied, and the resultant combined stress must be obtained (Art. 243).† Any

* Courtesy of the Shepard Electric Crane and Hoist Co., Montour Falls, N. Y.

† Some designers make assumptions for such work. They assume that the upper column load is carried by the portion of the lower column directly under it and proportion the remainder of the lower column section to resist the load from the crane girders. Any bending due to lateral or wind forces is assumed to be resisted by the whole section.

Fig. 606*

Fig. 607*

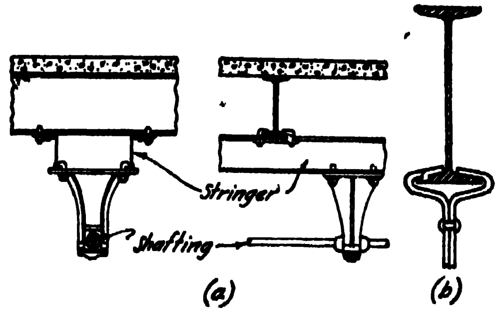


Fig. 608*

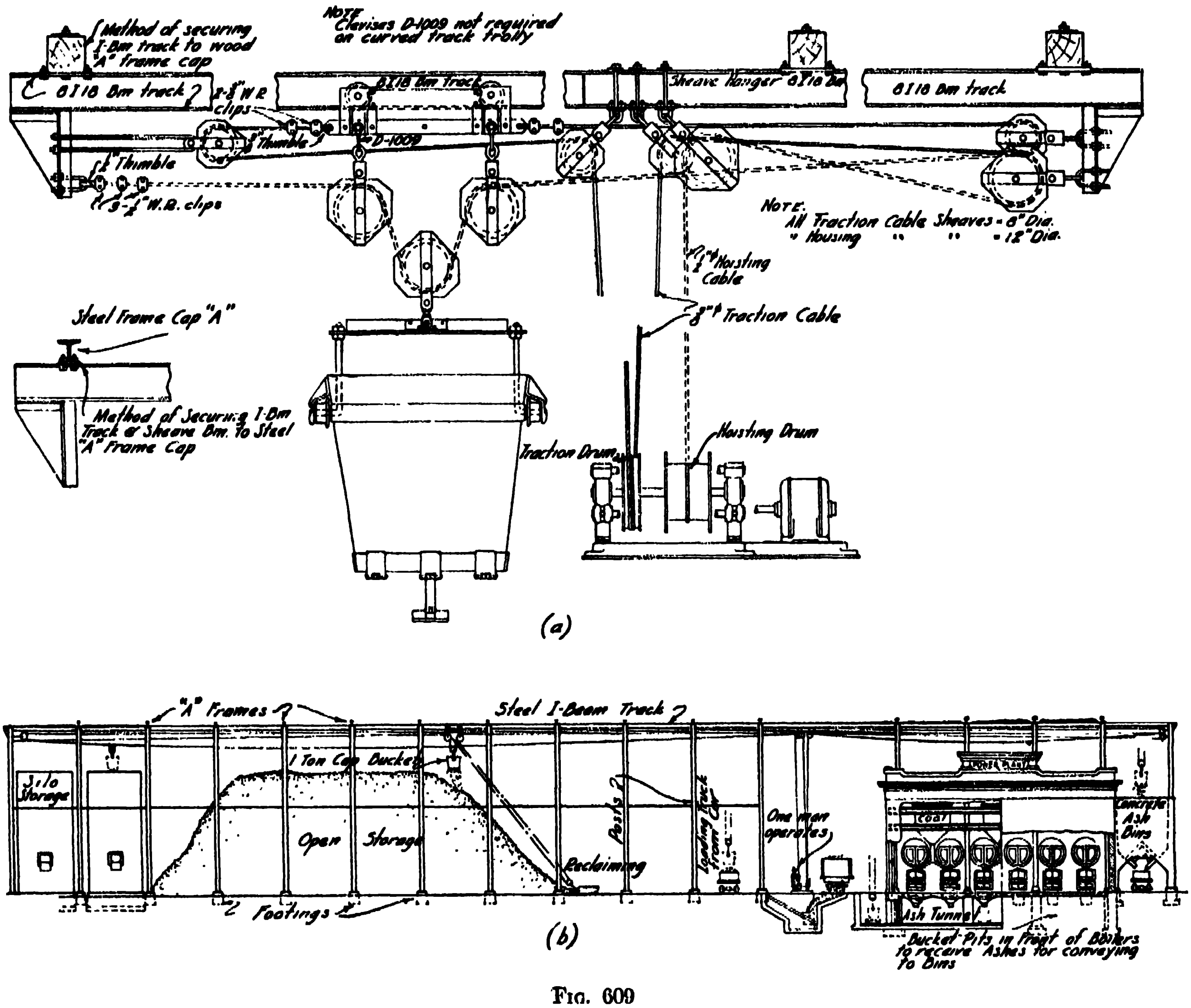


Fig. 609

* Courtesy of the Shepard Electric Crane and Hoist Co., Montour Falls, N. Y.

lateral forces due to wind must be considered (Art. 363) in determining the resultant moment, if they exist. As stated in Art. 370, a substantial diaphragm should be used to connect the upper section of the column to the crane girders, as the upper column should be strengthened to resist any lateral forces from the crane (see Fig. 599). Figure 603 shows crane girder connections to crane columns.

372. Trolley Beams.

In mill handling systems, overhead transportation is often supplied by trolley hoists which run on stationary I beam runways. The hoists are generally of 1 or 2 tons capacity, and the weight of the hoist is roughly about 500#. Figure 604 shows a typical installation with curved sections as well as straight ones, with switches. The wheels may run directly on the lower flanges of the I beam, as shown in Fig. 605 (*a*) and (*b*), or T rail clamps may be clamped to the flanges by means of bolts and spreader castings, as shown in Fig. 605 (*c*). While the I beam shape is ideally suited to runway work, the flanges are too soft to be used continuously in active service to carry the wheels, and the T rails were developed for this purpose (Fig. 606). In addition, no punching is required in the I beams, and better bearing is provided for the trolley wheels. Figure 607 shows a typical installation of the latter type with the trolley stop and incident structural framing, and the T-bar conductors for the electric current, the conductor supports, splices, and insulators near the top flange. The drum of the hoist may be mounted either parallel or at right angles to the track, — the former being common (Fig. 605 (*a*)). In some cases, trolley hoists are run on the bottom chords of trusses. Here, the bottom chords are made of a pair of channels back to back between which the lower gusset plates may be inserted.

In some instances, the structural engineer provides for the I beam track as a part of the structural steel work. In other cases, it is made a part of the mechanical equipment. Either case requires that provision be made for its installation and that the members which carry the I beam track are properly designed to carry the loads. The trolley beams may be designed by the usual methods, once the proper loads are obtained from the hoist manufacturer (load at mid-span for bending, and load at one end for shear).

373. Shafting Hangers.

In many cases, shafting for machinery must be carried by the structural frame, and allowances must be made for the loads. When the shafting runs at right angles to the beams, stringers may be used, as shown in Fig. 608 (*a*). When it runs parallel to the beams, the hangers may be attached directly to the beams or by the use of bolting blocks. Hangers for various other purposes may be employed also, such as for attaching an overhead motor, as in Fig. 608 (*b*). For ordinary cases, the beam when designed for reasonably heavy floor loads is sufficient for such random loads, and such investigations are commonly not made, — in some cases when they should be.

374. Miscellaneous Equipment Supports.

In many cases, the structural engineer must coöperate with the mechanical engineer in planning supports for many kinds of equipment, such as hoists, conveyors, and so on. Figure 609 illustrates two typical instances. While such problems are not ordinarily met with in architectural construction, yet the same principles of design may be employed.

INDEX

INDEX

(Numbers refer to pages)

T